CONCRETE
AND
CONSTRUCTIONAL ENGINEERING

INCLUDING PRESTRESSED CONCRETE

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FIFTIETH YEAR OF PUBLICATION

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LEADING CONTENTS

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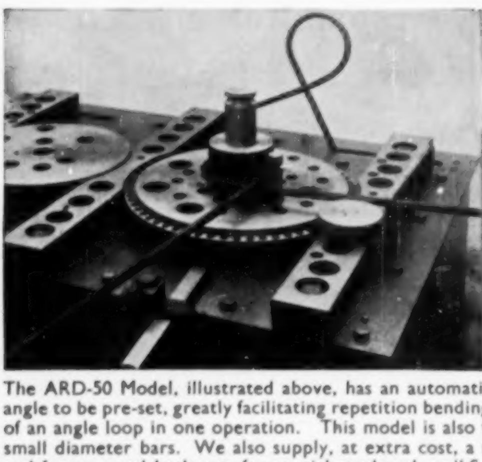

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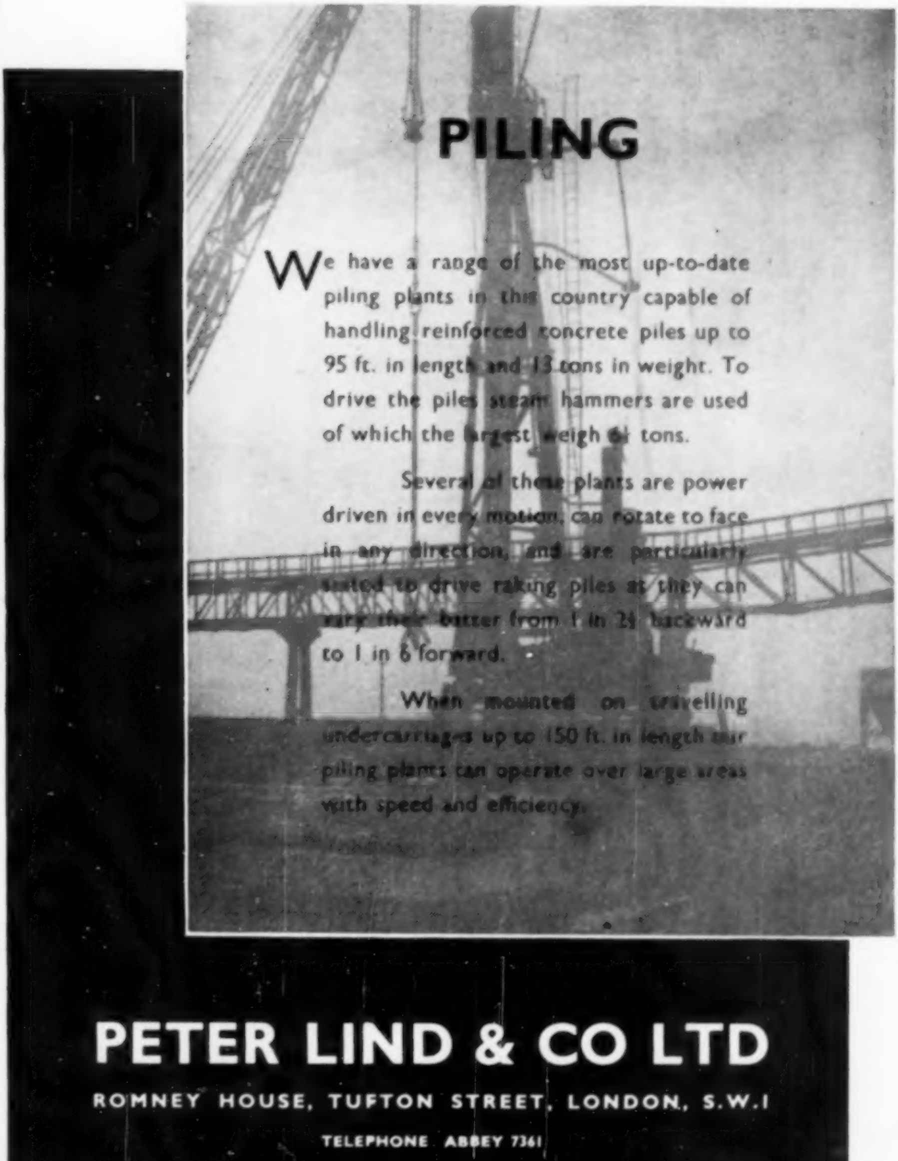
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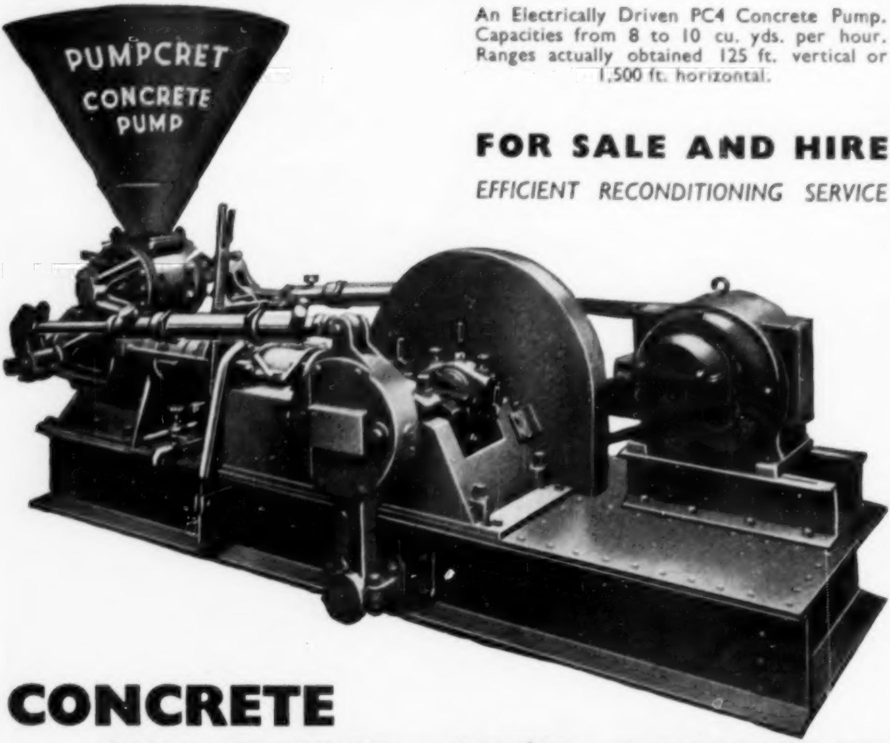

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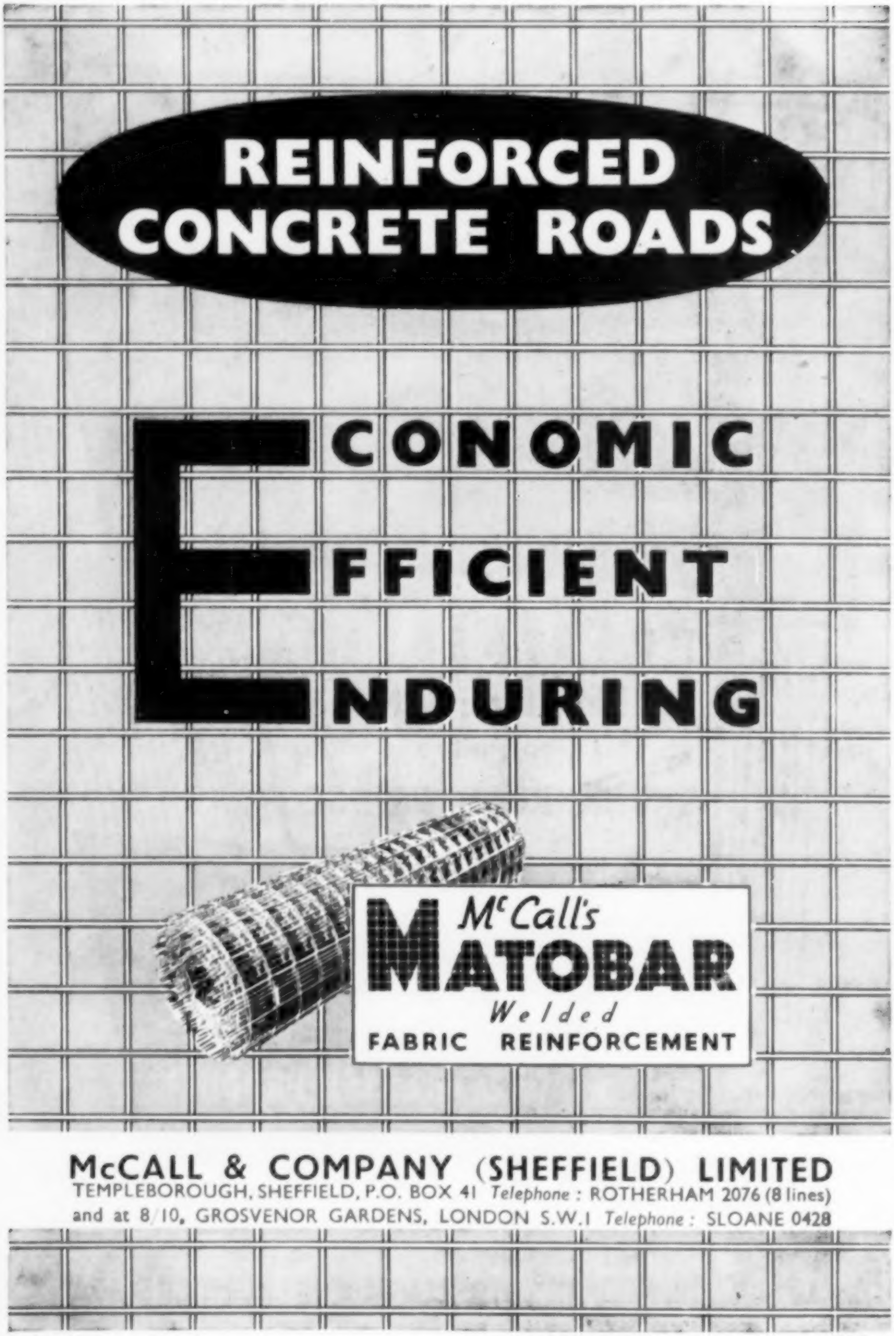
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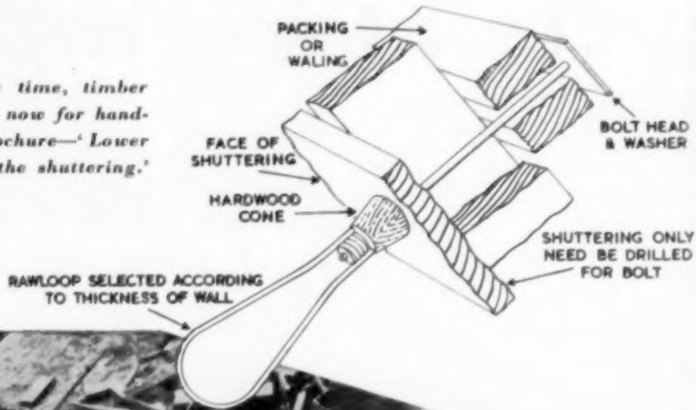

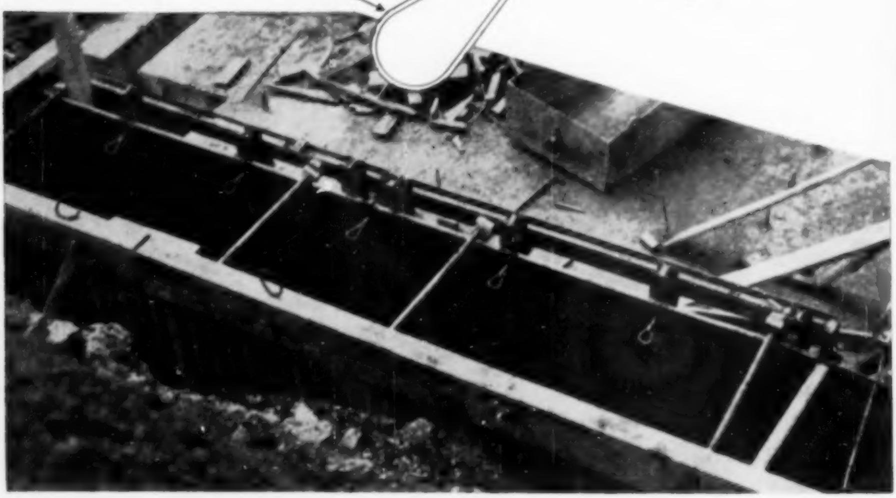

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CONCRETE
AND
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Volume L, No. 8. LONDON, AUGUST, 1955.

EDITORIAL NOTES

Liberal Education in a Technical Age.

NOTES in this journal during the past twenty years or so deploring the lack of education of many of the younger technologists have resulted in the receipt of many resentful letters. That the views we have expressed are not unfounded, however, is shown by the fact that the problem has in recent years increasingly been debated in Parliament and elsewhere, and much concern has been expressed at the tendency for men entering the professions to be technologists in one narrow branch of knowledge rather than what Bacon described as " whole men ". The latest inquiry into the problem is that conducted by a committee appointed by the National Institute of Adult Education, whose report * is now published. This report, like others before it, confirms all that we have written. It agrees that the possession of a General Certificate of Education in approved subjects by a child of sixteen years of age " does not amount to much ", although it is commonly the only standard of education required for the acceptance of a student of a technical subject. It agrees that cramming in a technical subject from that age onwards leaves no room for further general education, although one or two technical colleges make some attempt to influence their students to improve their general education. The standard of English of such students is agreed to be often deplorable, even in the case of men who have obtained a university degree, and attention is directed to the need to develop in men trained in science and technology the qualities that will fit them to deal with problems of policy, management, and human relations—or, it may be added, to make them good scientists and technologists.

The report emphasises the duty of teachers of technical subjects to pay attention to the students' use of English as an essential means of thought and communication. Many of the teachers who replied to a questionary agreed with this view, but others thought that their only duty was to teach a vocation in which the student could earn a living. It is deplorable that teachers should be content to accept such a narrow view of their responsibilities, but it may be that they themselves are not all capable of doing more. The need for precise language is as necessary in science and technology as it is in the law, yet how often do we find teachers who confuse their students by the misuse of simple words? It is, for example, fairly common to see and hear a load on a structure or a foundation

* " Liberal Education in a Technical Age." [London: Max Parrish & Co., Ltd. Price 6s.]

called a reaction, whereas in fact it is the opposite—it is the action or force that causes a reaction. A lecturer some time ago excused his misuse of this word on the ground that when he was a student it was so used by his lecturer, and neither of them knew that reaction was the opposite of action. In the Code of Practice for Foundations issued by the Institution of Civil Engineers last year the resistance of the earth under a foundation is called " earth pressure ", which is the opposite of the truth. Prestressing has also resulted in terms which are frequently used in the opposite sense of their true meaning. For example, prestressed members are often described as pre-tensioned, which can only mean that they are stretched before they are erected, whereas in fact the exact opposite is the case—they are compressed. A common excuse for this misuse of words is that " we all know what we mean by them ", but how stupid it is to confuse the mind by using ordinary words in the opposite sense of their true meaning in a technical school and in published matter on structural engineering. It should be remembered that large numbers of British technical publications are exported to countries where the readers know only the meanings given to words in a dictionary, and who are not told that the writer does not always say what he means—for example, that when he writes pressure he may mean resistance, that tensioned may mean compressed, that reaction may mean an applied force, and so on.

Last year a symposium was held in London on " Mix Design and Quality Control of Concrete ". Here the ungrammatical slang of the navvy is used in the title of the symposium and throughout many of the papers. In a philosophical discussion there is no excuse for using the verb " mix " variously as a noun and as an adjective. Indeed, we should be very stupid to mix design if we wished to design a mixture. Also, the misuse of the word quality as an adjective destroys the sense of the title ; what is meant is control of the quality of concrete rather than the quality of the control. The fact that the Department of Scientific and Industrial Research uses this bad English is not a good reason why others should do so ; official bodies should rather set an example.

The report places some of the responsibility on professional institutions that are content to be concerned only with technical knowledge in their examinations for membership, with the result that technical education is concentrated on the task of working for examinations. This problem was put by the committee to some of the professional institutions, but generally the reply was that they were concerned only with competence in " specialised techniques ". The committee's main recommendation is that the technical schools should pay more attention to giving a liberal education as well as technical training to future scientific and technological workers. It is suggested in this report that the time necessary for broader education be found by omitting some of the details now required at examinations but which need not be remembered afterwards, and to this end it is further suggested that students be allowed to use reference books at examinations to save them " memorising material which no sensible practitioner tries to carry in his head ". This is contrary to the present conception of the purpose of an examination, but it is an idea that may compare well with a system in which memory may be more important than ability, and by which men who cannot spell simple words or put their thoughts intelligibly on paper can obtain a university degree which should be a hallmark of education as well as of technical competence.

Design of Non-Prismatic Members.

A Numerical Method.

By H. P. VASWANI, B.Eng., D.I.C.

THE moment-distribution method requires the determination of carry-over factors and distribution factors at every joint. These can be easily calculated when the members have constant moments of inertia; the carry-over factor for such a member is 0·5 and its stiffness is proportional to its moment of inertia divided by its length. If the moment of inertia is not constant, strain-energy methods are often used. However, when the moment of inertia does not follow a simple mathematical curve, integration of the expressions obtained by strain-energy equations may be too involved to permit an easy solution, or sometimes may even be insoluble. In such cases numerical methods may be used. The application of relaxation methods was recently published.(1) In this article Simpson's rule is used to find the stiffnesses and carry-over factors of non-prismatic members.

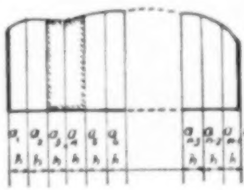


Fig. 1.

According to Simpson's rule the area A″ of the two adjacent strips shown shaded in *Fig.* 1 can be written as $A'' = \frac{h}{3}(a_3 + 4a_4 + a_5)$, where a_1, a_2, etc., are ordinates and h is the distance between the ordinates. Hence the area A of the diagram, divided into an even number of strips, is obtained by adding areas of successive strips and can be written

$$A = \frac{h}{3}(a_1 + 4a_2 + 2a_3 + 4a_4 \;.\;.\;.\; 2a_{n-2} + 4a_{n-1} + a_n).$$

Since the rule assumes that the ordinates a_1, a_2, a_3, etc., are joined by a second-degree parabola, for accurate results, only four intervals are necessary if the curve is parabolic, and only two if the curve is linear. For a smooth curve of general form Simpson's rule can be extended to give greater accuracy by including differences of a higher order in the Stirling formula, which result in the expression $A'' = \frac{h}{90}(-a_2 + 34a_3 + 114a_4 + 34a_5 - a_6)$ for the area of the two adjacent strips shown shaded in *Fig.* 1.

Simpson's rule can also be used to integrate one diagram with another (2), and hence can be applied to problems in structural analysis; here its use is extended to integrate three diagrams with one another.

Fig. 2 shows a beam AB of varying moment of inertia. The carry-over factor to B and the slope of the tangent at A are to be determined when a moment is

applied at A. A bending moment M_A is applied at A and it is assumed that A is pinned and B encastré. The ratio of the bending moment carried over (to B) to M_A is the carry-over factor. The span AB is divided into a convenient even number of short lengths n. The bending-moment diagram is drawn, and values $m_1, m_2, m_3 \ldots m_n$ are indicated in terms of M_A and M_B, where M_B is unknown. Similarly, diagrams for $\frac{I}{EI}$ and x are drawn and values $i_1, i_2, i_3 \ldots i_n$ in terms of I_0 and $x_1, x_2, x_3 \ldots x_n$ are shown in terms of l_0.

By area-moment theorem No. I, $\int \frac{M}{EI} dx = \theta_A$, that is the bending-moment diagram is to be integrated with the diagram for $\frac{I}{EI}$ and equated to θ_A; this may be rewritten

$$\frac{h}{3}(m_1 i_1 + 4m_2 i_2 + 2m_3 i_3 \ldots 4m_{n-1} i_{n-1} + m_n i_n) = \theta_A.$$

By area-moment theorem No. II, $\int \frac{Mx}{EI} dx = \delta_A = 0$, where x is measured from A. This equation may be rewritten

$$\frac{h}{3}(m_1 i_1 x_1 + 4m_2 i_2 x_2 + 2m_3 i_3 x_3 \ldots 4m_{n-1} i_{n-1} x_{n-1} + m_n i_n x_n) = 0.$$

From these two equations the relationship between M_A, M_B, and θ_A is derived

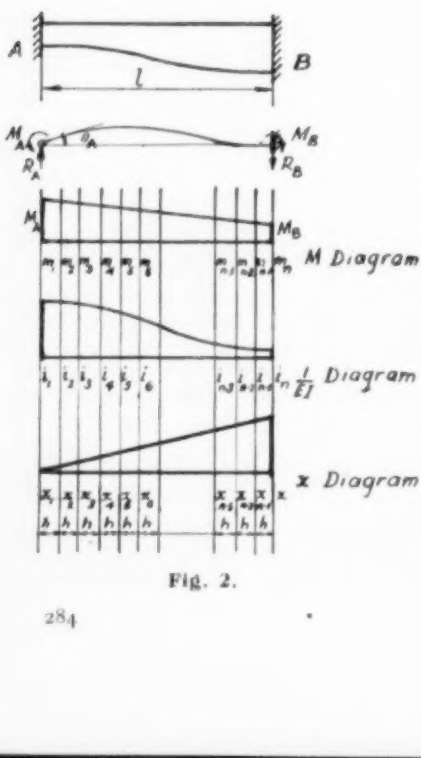


Fig. 2.

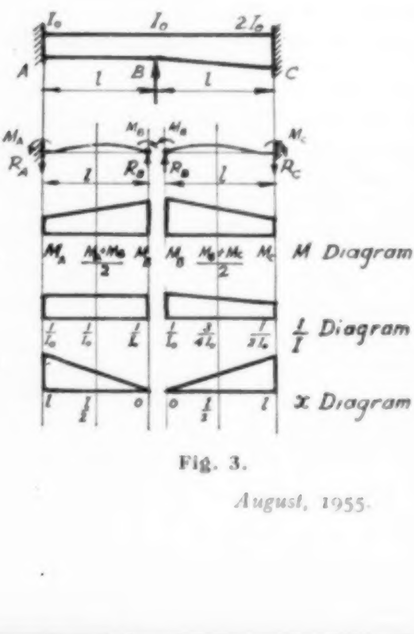


Fig. 3.

and is used in the following example to determine distribution factors for use in moment-distribution analyses.

EXAMPLE 1.—The end supports of a two-span continuous beam (*Fig.* 3) are encastré and the spans are of equal length. In the first span the moment of inertia has a constant value I_o; in the second span it varies from I_o at the central support to $2I_o$ at the encastré end. The stiffness at the end is defined as the moment required to rotate that end, when it is assumed to be simply supported, through a unit angle when the other end is fixed.

In the span AB the applied bending moment M_B is such that the tangent at B rotates through an angle θ_B when B is assumed to be simply supported. Then the area of $\frac{M}{EI}$ diagram, $\int \frac{M}{EI}dx = \theta_B = 1$, is

$$\frac{M_A + M_B}{2} \cdot \frac{l}{EI_o} = 1 \qquad (1)$$

The deflection δ_B measured from the tangent at A $= \int \frac{Mx}{EI}dx = 0$, where x is measured from B, is

$$\frac{l}{2} \cdot \frac{1}{3E}\left\{\frac{M_A l}{I_o} + 4 \cdot \frac{M_A + M_B}{2} \cdot \frac{l}{2} \cdot \frac{1}{I_o}\right\} = 0 \qquad (2)$$

From (1) and (2), $M_B = \frac{4EI_o}{l}$ is the stiffness of BA at B.

Similarly for beam BC,

$$\frac{l}{2} \cdot \frac{1}{3E}\left\{\frac{M_C}{2I_o} + 4\frac{M_C + M_B}{2} \cdot \frac{3}{4I_o} + \frac{M_B}{I_o}\right\} = 1 \qquad (3)$$

$$\frac{l}{2} \cdot \frac{1}{3E}\left\{\frac{M_C l}{2I_o} + 4\frac{M_C + M_B}{2} \cdot \frac{3}{4I_o} \cdot \frac{l}{2}\right\} = 0 \qquad (4)$$

From (3) and (4), $M_B = \frac{60}{13} \cdot \frac{EI_o}{l}$ is the stiffness of BC at B.

Hence the distribution factors are, for BA, $\left[\frac{4EI_o}{l}\right] \div \left[\frac{4EI_o}{l} + \frac{60}{13} \cdot \frac{EI_o}{l}\right] = 0{\cdot}465$ and, for BC, $1 - 0{\cdot}465 = 0{\cdot}535$.

EXAMPLE 2.—Find the fixed-end moments in the encastré beam (*Fig.* 4) whose moment of inertia varies from I_o at A to $2I_o$ at B. The beam has a uniformly-distributed load of intensity w.

From area-moment theorem No. 1, $\int \frac{M}{EI}dx = 0$, that is the area of the $\frac{M_o}{EI}$ diagram is equal to the area of the $\frac{M_F}{EI}$ diagram. Integrating the M_o diagram with the $\frac{1}{EI}$ diagram,

$$\frac{l}{4} \cdot \frac{1}{3E}\left\{4 \cdot \frac{7}{8I_o} \cdot \frac{3M_o}{4} + 2 \cdot \frac{3}{4I_o} \cdot M_o + 4 \cdot \frac{5}{8I_o} \cdot \frac{3}{4}M_o\right\} = \frac{1}{2}\frac{M_o l}{EI_o} = \frac{wl^3}{16EI_o}.$$

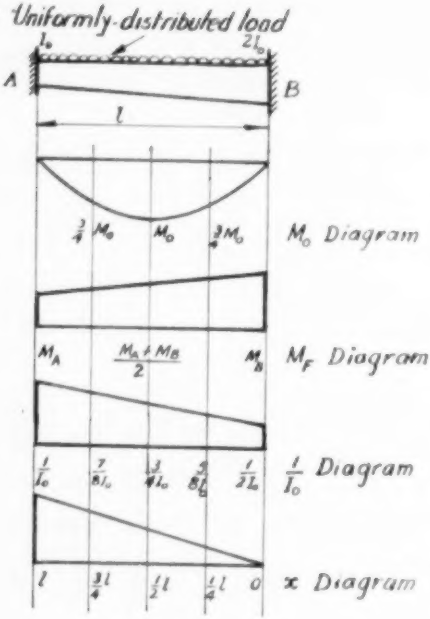


Fig. 4.

Integrating the M_F diagram with the $\frac{1}{EI}$ diagram,

$$\frac{l}{2}\cdot\frac{1}{3E}\left\{\frac{M_A}{I_o} + 4\cdot\frac{M_A+M_B}{2}\cdot\frac{3}{4I_o} + \frac{M_B}{2I_o}\right\} = \frac{5}{12}\cdot\frac{M_A}{EI_o}.l + \frac{1}{3}\cdot\frac{M_B}{EI_o}.l.$$

Therefore
$$\frac{5}{12}\cdot\frac{M_Al}{EI_o} + \frac{1}{3}\cdot\frac{M_Bl}{EI_o} = \frac{wl^3}{16EI_o} \quad . \quad . \quad . \quad . \quad . \quad (5)$$

From area-moment theorem No. II, $\int\frac{Mx}{EI}dx = 0$. Integrating the M_o diagram with the $\frac{1}{EI}$ and x diagrams, where x is measured from B,

$$\frac{l}{4}\cdot\frac{1}{3E}\left\{4\cdot\frac{7}{8I_o}\cdot\frac{3}{4}M_o\cdot\frac{3}{4}l + 2\cdot\frac{3}{4I_o}\cdot M_o\cdot\frac{l}{2} + 4\cdot\frac{5}{8I_o}\cdot\frac{3}{4}M_o\cdot\frac{l}{4}\right\} = \frac{17}{64}\cdot\frac{M_ol^2}{EI_o} = \frac{17}{512}\cdot\frac{wl^4}{EI_o}.$$

Integrating the M_F diagram with the $\frac{1}{EI}$ and x diagrams, where x is measured from B,

$$\frac{l}{2}\cdot\frac{1}{3E}\left\{\frac{M_A}{I_o}.l + 4\cdot\frac{M_A+M_B}{2}\cdot\frac{3}{4I_o}\cdot\frac{l}{2}\right\} = \frac{7}{24}\cdot\frac{M_Al^2}{EI_o} + \frac{3}{24}\cdot\frac{M_Bl^2}{EI_o}.$$

Therefore
$$\frac{7}{24}\cdot\frac{M_Al^2}{EI_o} + \frac{3}{24}\cdot\frac{M_Bl^2}{EI_o} = \frac{17}{512}\cdot\frac{\omega_o l^4}{EI_o} \quad . \quad . \quad . \quad . \quad (6)$$

From (5) and (6), $M_A = \frac{15}{208}.wl^2$ and $M_B = \frac{20{\cdot}25}{208}.wl^2$.

[REFERENCES: (1) E. Markland. "The Calculation of Elastic Properties of Non-Prismatic Structural Members." Civ. Eng. & P.W. Review, Vol. 49, No. 571, January 1954. (2) A. L. L. Baker. "Further Research in Reinforced Concrete, and its Application to Ultimate Load Design." Proc. I.C.E., Aug. 1953 (No. 2), Vol. 2, Part III.]

The late Professor Gustave Magnel.

It is with profound regret that we record the death of Professor Gustave Magnel, which occurred suddenly in Ghent on July 5, at the age of 66, as the result of a heart attack. By his passing the world loses one of its greatest civil engineers, whose specialisation in reinforced concrete and prestressed concrete had a great influence in the development of these new materials during the past thirty years.

After graduating at Ghent University, he came to England in 1914 and was one of the band of Continental engineers who were employed by the late D. G. Somerville for the primary purpose of training British graduates at a time when little attention was given to reinforced concrete at British universities and when text-books were few. Somerville's far-sighted effort in using his office as a well-paid post-graduate course was of inestimable value to the men trained in his office and who remember with gratitude both him and Magnel, who soon became Somerville's chief engineer. At one time, indeed, Somerville had the temerity to open an office in Paris for the design and erection of reinforced concrete structures, and Magnel took charge of this office.

In 1919 Magnel returned to the University at Ghent, where he soon became Professor of Reinforced Concrete. He had a gift for lucid exposition, and was a great teacher. His books "Cours de Stabilité des Constructions," "Pratique du Calcul du Béton Armé," and "Le Calcul Pratique des Poutres Vierendeel" are masterly works that are treasured by many engineers outside Belgium. One of his activities was to develop the laboratory at Ghent until it was one of the best in the world. He had little use for tests on small-scale models, and whenever possible insisted on testing full-size members, either in the laboratory or on the site.

During the last war, when Belgium was occupied and he had no access to outside information, he studied the possibilities of prestressed concrete and produced his well-known "Belgian" system. Magnel soon became an enthusiastic advocate of the new method of construction, and since the war his system has been largely used in many countries of the world. Like his other published works, his book "Prestressed Concrete" is notable for its clear exposition and avoidance of all that had not in his opinion been proved to be sound. It is typical of his outlook on engineering that in this book, while other systems are adequately described, all the examples of prestressed concrete

work are structures that he designed himself or whose design and construction he had supervised. He preferred to write only of what he knew and to take nothing at second hand, and as he had designed and built most types of structures in prestressed concrete the book did not suffer.

His last effort was to design a prestressed concrete tower more than 2000 ft. high (about twice the height of the Eiffel tower) for the Brussels World Fair of 1958, and when he showed us the drawings a few months ago he said that he would retire when this was built. His tower was to be a circular shaft about 200 ft. diameter at the bottom and tapering towards the top, and would be built of precast members cast on the site. There was to be a central lift-shaft and rooms for letting on the 200 or so floors. The top ten stories were to be used for the broadcasting of television programmes and the rest would be let for other purposes. The Belgian Government is now considering the project, which would cost about £3,000,000.

Magnel was very proud to have been elected a Member of the Belgian Royal Academy, and was intensely patriotic; during the war he was imprisoned by the Germans for his refusal to collaborate with them. His greatest pleasure was to train good Belgian engineers who would work in other countries and add to the prestige of his country, and it is notable that he gave the name of his country, rather than his own, to his system of prestressed concrete. He did much engineering work for the Belgian Government. In 1945 he was elected one of the first members of the United Nations Educational, Scientific and Cultural Organisation. He will be mourned by a host of friends throughout the world, who will sadly miss his genial presence and his never-failing helpfulness.

Book Reviews.

"Data Book for Civil Engineers." Vol. 3, "Field Practice." By Elwyn E. Seelye. (London: Chapman and Hall, Ltd. Second edition. 1954. Price 60s.)

THIS is one of a series of three American books of data for civil engineers. Part I (277 pages) deals with the problems and duties of inspectors of works. Part II (105 pages) gives comprehensive notes, tables, and formulæ on surveying. Transition curves and spirals are fully dealt with, and the tables for the measurement of earthworks are particularly useful.

Part II is divided into sections, and the largest (58 pages) describes the examination, classification, and testing of subsoils. The bearing resistance of soils is discussed, but there is little information about earth pressure, embankments, and cuttings. Most of the work described would normally be carried out to a specification written by an engineer experienced in this class of work, so that much of the data appears to be unnecessary; some of this space could have been usefully given to notes on temporary site works in timber.

Other sections deal with equipment and construction methods, concrete, masonry, structural steel, timber, bridges, piles, bituminous paving, and sewage and drainage works. Each section includes a list of items for the inspector to check and also gives examples of report sheets. In the section on concrete are notes on the testing of aggregates, reinforcement, concreting in cold weather, and air-entrained concrete. The data on concrete mixtures are ample, and the tables relating the weight of concrete to its probable strength are useful. The "Engineering News" pile-driving formulæ are given, but tables of allowable loads are based on modified formulæ.

"Solution of Problems in Surveying and Field Astronomy." By H. W. Stephenson. (London: 1955. Sir Isaac Pitman & Sons, Ltd. Price 25s.)

THIS is a collection of problems in surveying, geodesy, and field astronomy based generally upon questions set in the examinations of the University of London and the Institution of Civil Engineers. In addition to the problems for which there are completely worked solutions, there are many to which the answers only are given. The book should be useful to students and to engineers in practice.

A New Method of Prestressing Circular Reservoirs.

An unusual method of constructing circular reservoirs with a capacity greater than about 500,000 gallons was described by MM. Marcel and André Reimbert in the French journal "Travaux" for October, 1953, from which the following is abstracted.

The wall (*Fig.* 1) comprises two leaves of which the inner is a series of horizontal arches of small radius and the outer is polygonal, the springings of the arches and the ends of the outer facets being monolithic (*Fig.* 2). The radius of the centre-line of an arch is about 3 ft. and the "rise" is one-third of the radius. The length of each facet of the external leaf is about 5 ft. 2 in. Steel shutters (*Fig.* 3) were used.

The wall has a sliding joint at the base so that direct stresses only are caused by the pressure of the contained liquid. The reinforcement in the arches and the secondary reinforcement is mild steel; the horizontal reinforcement which acts as a tie in the outer part of the wall is of high-tensile steel.

Fig. 3.—Reinforcement and Shutters for Part of Wall.

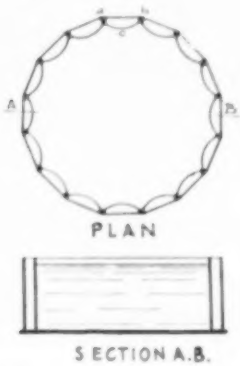


Fig. 1.—Plan and Section.

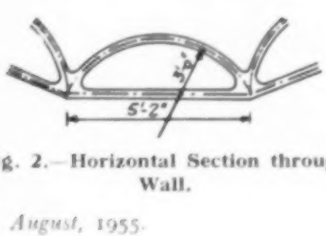


Fig. 2.—Horizontal Section through Wall.

Construction.

The reservoir is constructed in four stages (*Fig.* 4). First the floor is laid on weak concrete placed over the site. Secondly a layer of bituminous material is placed around the periphery to form a sliding joint for the wall, and on top of this the concrete base of the wall is cast with reinforcement projecting to lap with the reinforcement in the first lift of the wall. The reinforcement is made up in cages (*Fig.* 5). Thirdly the wall is cast to the full height (*Fig.* 6). This may be done in either of two ways: (1) After the reinforcement is placed the arches only may be cast as shown in *Fig.* 7*a* with the ties left exposed between the springings, or (2) the arches and the external polygonal leaf are cast (*Fig.* 7*b*) leaving a gap of about 6 in. in the centre of each facet; in this case the tie-bars passing through the outer part of the wall are painted with bitumen before the concrete is cast. The bituminous joint between the toe of the wall and the floor is also made at this time. When the concrete in the wall and

floor is sufficiently strong the reservoir is filled and the increase in diameter due to the internal pressure elongates the tie-bars. The remainder of the external part of the wall is then concreted and is carefully cured to reduce shrinkage and creep. When this concrete is sufficiently strong the reservoir is emptied and the tie-bars compress the concrete in the outer part of the wall due to their elastic recovery as the hydraulic pressure is reduced. Thus when the tank is refilled the arches are in compression and there is no tensile stress in the concrete in the outer part of the wall.

The compressive stress in the arches is small. Consider for example a single arch (*Fig.* 8). If w is the density of the liquid, H the height of liquid above the point considered, t the thickness of the arch, and r the radius of curvature of the centre-line of the arch, the compressive force in the arch is wHr and the stress f is $\frac{wHr}{t}$, so that with a reservoir 30 ft. high in which the arches are $2\frac{1}{2}$ in. thick and have a radius of 3 ft. $f = \frac{62{\cdot}5 \times 30 \times 3}{2{\cdot}5} = 225$ lb. per square inch.

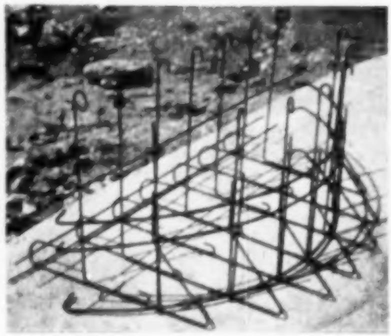


Fig. 5.—Reinforcement at Base of Wall.

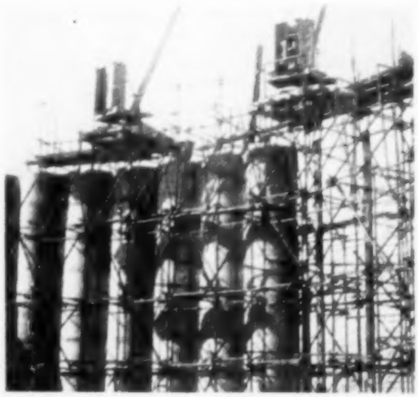


Fig. 6.—Casting the Walls.

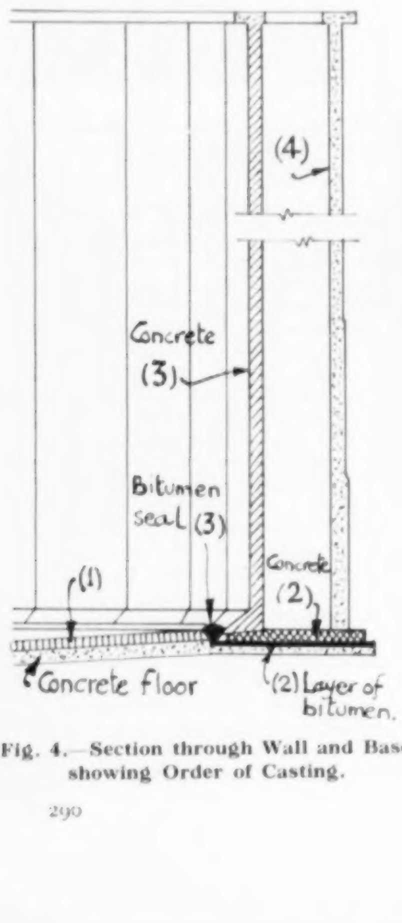


Fig. 4.—Section through Wall and Base showing Order of Casting.

Comparison of Quantities of Materials.

The following example is given to show the quantities of materials required for a reservoir designed by this method compared with the quantities required for a reservoir of the same capacity designed

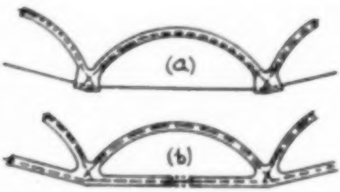


Fig. 7.

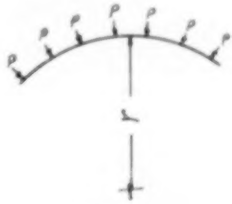


Fig. 8.

in the usual manner. The reservoir has a capacity of 1,770,000 gal. and is 26 ft. 3 in. high. The quantities for the common type of reservoir are given first. Concrete in wall and floor, 612 cu. yd. (342 cu. yd.); reinforcement, 59 tons (7·9 tons of mild steel and 14·125 tons of high-tensile steel); internal waterproof lining, 2300 sq. yd. (2540 sq. yd.). It is stated that the cost of a reservoir built in the manner described is less than one of the usual type, and that by the use of precast arches a further saving can be made.

Compacting Soil by Vibration.

THE machine shown in *Fig.* 1 is one of a series developed by Vibro-Plus Products, Inc., of Woodside, N.Y., U.S.A., for compacting soils. The machine illus-

Fig. 1.

trated weighs 3 tons, and is driven by a 25-h.p. diesel engine. The front cylinder is 4 ft. 10 in. long by 4 ft. diameter, and is vibrated at variable frequencies between 1400 and 1600 vibrations per minute; at maximum frequency the centrifugal force of the vibration is about 7 tons. Tests have shown that three passes of the roller over silty sand produced a compaction of 95 per cent. to a depth of 1 ft. and that six passes produced 100 per cent. compaction at that depth. Two passes at a speed of two miles per hour produced 95 per cent. density to a depth of 2 ft. in the case of soil comprising 80 per cent. of fine sand, 12 per cent. of coarse sand, and 8 per cent. of silt and clay.

Heavy Loads on Piles.

A BRIDGE in California, U.S.A., is to be carried on 238 piles each designed to carry a load of 225 tons. The piles are H-shape in cross section. A 14-in. pile of this shape, 138 ft. long, was driven to a solid bearing, and loaded with 45 tons. The total settlement under this load was ¾ in., and the settlement on removal of the load was ¼ in. This is thought to be the greatest load ever imposed on a pile of this shape. The contractors are the Duncanson-Harrelson Co. and the Pacific Bridge Co. The method of loading the pile is described in "Engineering News-Record" for February 24, 1955.

"Lift-up" Floor Construction.

THE "lift-up" method of floor construction is being used for floors with a total area of 215,000 sq. ft. in three-story barracks in Virginia for the United States Navy. The same system is being used at other large barracks now in course of construction in the U.S.A. The saving in cost compared with in-situ floors is estimated to be nearly 10 per cent.

At the Virginia barracks (*Fig.* 1) the ground-floor slab was laid first, and on this were cast the two upper floors and the roof. The wall columns are each formed of two 8-in. by 8-in. by ½-in. steel angles and are spaced at 22-ft. centres transversely and at 20-ft. centres longitudinally. When the columns were erected the side-forms were fixed for the ground floor, the reinforcement and all pipes and services were laid in position, and the slab was concreted. Eighteen hours later the top of this slab was coated with a preparation with a base of paraffin. Lifting collars were fixed to the steel columns, and the slab for the first floor was concreted. The slabs for the second floor and roof were then concreted in the same manner, each separated from the next by paraffin emulsion. Each slab measured 53 ft. by 76 ft., and was lifted by the collars on twelve columns.

An hydraulic jack was placed on the top of each column, and from each jack a threaded rod extended on either side of the column to the lifting collars, to which they were fixed by lugs. The jacks were operated and the slabs were raised at the rate of about 5 ft. per hour. When each slab arrived at the required height, steel blocks were welded to the columns below the collars to support the slab. When the roof slab was fixed, the lugs were removed from the threaded rods and lowered for use in lifting the next slab. The first and second floors are 9 in. thick and the roof 7½ in. thick. They are reinforced in the normal manner. The height of each floor in these structures is about 11 ft.

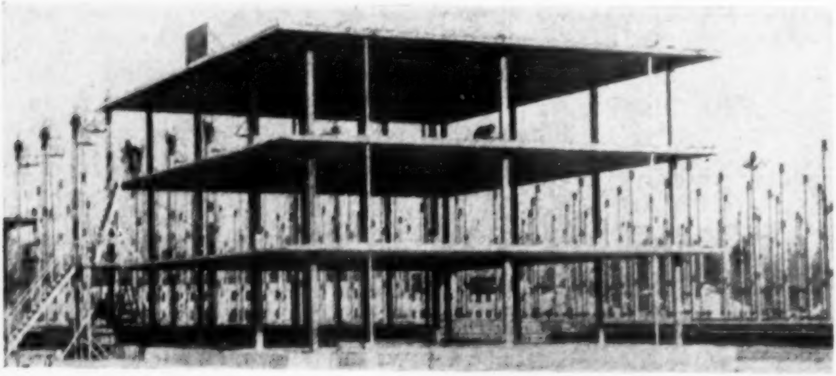


Fig. 1. Floors Cast at Ground Level and Hoisted into Position.

The slabs were raised by N.Y. Lift Slabs, Inc., of San Antonio, Texas.

Bending and Axial Stresses in Circular Columns.

By R. J. BARTLETT.

THE following notes describe a method of analysing the stresses in a circular reinforced concrete column in which tensile stresses are caused by a bending moment M combined with an axial load W. It is assumed that the load W acts at the centre of the section and that the bending moment acts about any vertical plane through the centre (*Fig.* 1). The reinforcement should be arranged symmetrically about the centre of the column, but otherwise its position relative to the direction in which the bending moment acts and the positions of the neutral axis and the point of maximum stress in the concrete are immaterial. The maximum stress in the steel is affected only by the distance of the reinforcement farthest from the neutral axis.

If the column is subjected to more than one bending moment, these may be combined vectorially into one bending moment and this method used without further correction. The neutral axis will be at right angles to the plane of the resultant bending moment.

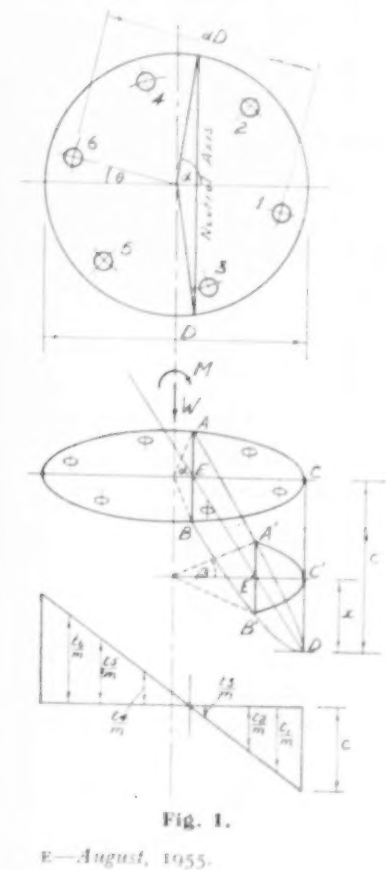


Fig. 1.

Method of Application.

Calculate the following: d, the ratio of the diameter of the circle in which the bars are placed to the diameter D of the column; e from $\frac{M}{WD}$; r, the ratio of the area of the reinforcement to the cross-sectional area of the column; and L from $\frac{\pi mrd^2}{4e}$.

Plot on graphs (*Fig.* 2) the following values:

ϕ	y
−0·5	$\frac{0{\cdot}0543}{e} + L$
0	$\frac{0{\cdot}1964}{e} + L$
0·5	$\frac{0{\cdot}3385}{e} + L$
0·7	$\frac{0{\cdot}3695}{e} + L$
1·1	$\frac{0{\cdot}3914}{e} + L$
1·5	$\frac{0{\cdot}3927}{e} + L$

Join these points together with straight lines; the true line joining these points is of such small curvature that for practical purposes a series of straight lines is a satisfactory substitute. Find the coordinates ϕ and y of the point at which the line cuts the curve of the appropriate ratio of r.

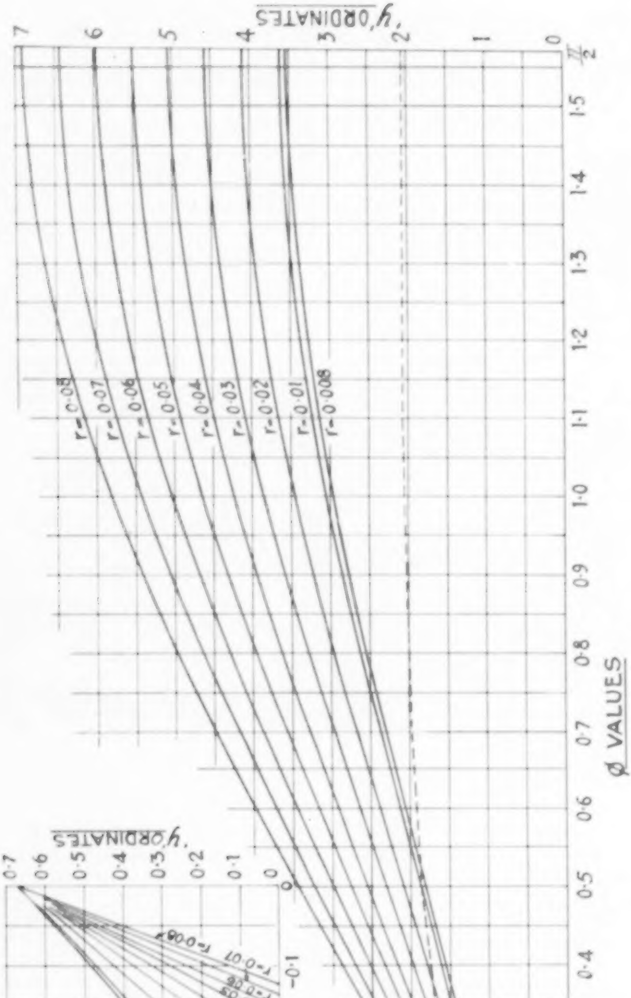

y, ORDINATES
ϕ VALUES
r=0·08
r=0·07
r=0·06
r=0·05
r=0·04
r=0·03
r=0·02
r=0·01
r=0·008
0
0·1
0·2
0·3
0·4
0·5
0·6
0·7
0·8
0·9
1·0
1·1
1·2
1·3
1·4
1·5
π/2
1
2
3
4
5
6
7
y, ORDINATES
ϕ VALUES
-0·4
-0·3
-0·2
-0·1
0
0·1
0·2
0·3
0·4
0·5
0·6
0·7
r=0·008
r=0·01
r=0·02
r=0·03
r=0·04
r=0·05
r=0·06
r=0·07
r=0·08

Fig. 2.

Then the maximum stress in the concrete c is

$$\frac{4W(1+\sin\phi)}{D^2\times y},$$

and the maximum stress in the steel t is

$$\frac{cm(d\cos\theta-\sin\phi)}{1+\sin\phi}$$

where θ is the angle between the plane of the bending moment and the diameter of the column drawn through the bar farthest from the neutral axis in the tensile side of the column.

Since only that part of the line which cuts the curve is necessary, it need be drawn only between the two values of ϕ concerned. There will be only tensile stresses if e exceeds about 0·125, and as this value increases the value ϕ will decrease from a maximum of $\frac{\pi}{2}$ to a negative quantity when e is infinity, that is when W is zero.

EXAMPLE.—A column of 12 in. diameter reinforced with six ¾-in. bars having 1½-in. cover is subjected to an axial load of 30,000 lb. and a bending moment of 90,000 in.-lb.

$$d=\frac{8\cdot25}{12}=0\cdot6875;$$

$$e=\frac{90{,}000}{30{,}000\times12}=0\cdot25;$$

$$r=\frac{2\cdot65\times4}{3\cdot14\times12^2}=0\cdot0234;$$

$$L=\frac{3\cdot14\times15\times0\cdot0234\times0\cdot6875^2}{4\times0\cdot25}=0\cdot52.$$

Value of ϕ	Values of y
−0·5	$\frac{0\cdot0543}{0\cdot25}+0\cdot52=0\cdot737$
0	$\frac{0\cdot1964}{0\cdot25}+0\cdot52=1\cdot305$
0·5	$\frac{0\cdot3385}{0\cdot25}+0\cdot52=1\cdot873$
0·7	$\frac{0\cdot3695}{0\cdot25}+0\cdot52=1\cdot998$
1·1	$\frac{0\cdot3914}{0\cdot25}+0\cdot52=2\cdot087$
1·5	$\frac{0\cdot3927}{0\cdot25}+0\cdot52=2\cdot090$

By plotting on the graph the values of y for $\phi=0$ and $\phi=0\cdot5$ only and joining them with a straight line it is found that a curve representing r equals 0·0234, interpolated between those for r equals 0·02 and 0·03, is cut by the line when ϕ is 0·347 and y is 1·7. Therefore

$$c=\frac{4\times30{,}000(1+\sin0\cdot347)}{144\times1\cdot7}=658\text{ lb. per sq. in.}$$

If θ equals $\frac{\pi}{6}$ (30 deg.), then

$$t=\frac{658\times15\left(0\cdot6875\cos\frac{\pi}{6}-\sin0\cdot347\right)}{1+\sin0\cdot347}$$

$= 1875$ lb. per sq. in.

Appendix.

If AB is the neutral axis in a circular section resulting from the application of an axial load W and a bending moment M, then the total compressive force in the concrete is equal to the volume of the ungula ABCD (*Fig.* 1) of a circular prism where CD represents the maximum compressive stress in the concrete.

If the boundary ACB of the area being compressed subtends an angle 2α at the centre, then the area of ACB is

$$\frac{D^2}{4}(\alpha-\sin\alpha\cos\alpha)$$

and the depth of the neutral axis EC is $\frac{D}{2}(1-\cos\alpha)$. The depth E′C′ of any plane A′C′B′ distant x from D and subtending an angle 2β at the centre is

$$\frac{D}{2}(1-\cos\beta),\text{ or }\frac{xD}{2c}(1-\cos\alpha).$$

Therefore $\cos\beta=1-kx$, where

$k=\frac{1-\cos\alpha}{c}$, and, by differentiating,

$$-\sin\beta d\beta=-k\,dx.$$

$$\text{Area A′C′B′}=\frac{D^2}{4}(\beta-\sin\beta\cos\beta).$$

Therefore volume ABCD

$$=\int_0^c\frac{D^2}{4}(\beta-\sin\beta\cos\beta)dx$$

$$=\frac{D^2}{4k}\int_0^\alpha(\beta-\sin\beta\cos\beta)\sin\beta d\beta$$

$$=\frac{D^2}{4k}\left[\sin\alpha-\alpha\cos\alpha-\frac{\sin^3\alpha}{3}\right]$$

$$= \frac{D^2c\left(\sin\alpha - \alpha\cos\alpha - \frac{\sin^3\alpha}{3}\right)}{4(1-\cos\alpha)}.$$

Since, in most cases, α exceeds $\frac{\pi}{2}$ it is convenient to substitute $\frac{\pi}{2} + \phi$ for α. Therefore volume ABCD

$$= \frac{D^2c}{4(1-\cos\alpha)}\left(\frac{3}{4}\sin\alpha - \alpha\cos\alpha + \frac{\sin 3\alpha}{12}\right)$$

$$= \frac{D^2c}{4(1+\sin\phi)}\left[\frac{3}{4}\cos\phi + \left(\frac{\pi}{2}+\phi\right)\sin\phi - \frac{\cos 3\phi}{12}\right]$$

$$= \frac{D^2cY}{4(1+\sin\phi)} \quad . \quad . \quad . \quad . \quad (1)$$

where Y is the expression between the square brackets.

The moment of the area ACB about the centre

$$= \frac{D^3}{4}\int_0^\alpha \cos\alpha\sin^2\alpha\, d\alpha = \frac{D^3}{12}\sin^3\alpha.$$

Therefore the moment of any plane A'C'B' about the centre $= \frac{D^3}{12}\sin^3\beta$, and the moment of the volume ABCD about the centre

$$= \frac{D^3}{12}\int_0^c \sin^3\beta\, dx = \frac{D^3}{12k}\int_0^\alpha \sin^4\beta\, d\beta$$

$$= \frac{D^3c}{96(1-\cos\alpha)}\left(3\alpha - 2\sin 2\alpha + \frac{\sin 4\alpha}{4}\right)$$

$$= \frac{D^3c}{96(1+\sin\phi)}\left[\frac{3\pi}{2} + 3\phi + 2\sin 2\phi + \frac{\sin 4\phi}{4}\right]$$

$$= \frac{D^3cX}{96(1+\sin\phi)} \quad . \quad . \quad . \quad . \quad (2)$$

where X is the expression between the square brackets.

From the geometry of the section and the stress-variation diagram, the stresses in each bar are related to the maximum compressive stress as follows:

$$\frac{c}{\frac{D}{2}(1-\cos\alpha)} = \frac{t_1}{\frac{mD}{2}(d\cos\theta - \cos\alpha)}$$

$$= \frac{t_2}{\frac{mD}{2}\left(d\cos\overline{\frac{\pi}{3}-\theta} - \cos\alpha\right)}$$

$$= \frac{t_3}{\frac{mD}{2}\left(d\cos\overline{\frac{\pi}{3}+\theta} - \cos\alpha\right)}$$

$$= \frac{t_4}{\frac{mD}{2}\left(d\cos\overline{\frac{\pi}{3}+\theta} + \cos\alpha\right)}$$

$$= \frac{t_5}{\frac{mD}{2}\left(d\cos\overline{\frac{\pi}{3}-\theta} + \cos\alpha\right)}$$

$$= \frac{t_6}{\frac{mD}{2}(d\cos\theta + \cos\alpha)}.$$

The cross-sectional area of each bar is $\frac{r\pi D^2}{24}$. The total compressive force minus the total tensile force in the bars is

$$\frac{r\pi D^2}{24}(t_1 + t_2 + t_3 - t_4 - t_5 - t_6)$$

$$= \frac{r\pi D^2mc}{24(1-\cos\alpha)}\Big(d\cos\theta - \cos\alpha + d\cos\overline{\frac{\pi}{3}-\theta} - \cos\alpha + d\cos\overline{\frac{\pi}{3}+\theta} - \cos\alpha - d\cos\overline{\frac{\pi}{3}+\theta} - \cos\alpha - d\cos\overline{\frac{\pi}{3}-\theta} - \cos\alpha - d\cos\theta - \cos\alpha\Big)$$

$$= \frac{r\pi D^2mc}{24(1-\cos\alpha)}(-6\cos\alpha)$$

$$= \frac{r\pi D^2mc\sin\phi}{4(1+\sin\phi)} \quad . \quad . \quad . \quad . \quad (3)$$

The moments of the forces in the bars about the centre of the section are

$$\frac{r\pi D^2}{24}\Big(t_1\frac{dD}{2}\cos\theta + t_2\frac{dD}{2}\cos\overline{\frac{\pi}{3}-\theta} + t_3\frac{dD}{2}\cos\overline{\frac{\pi}{3}+\theta} + t_4\frac{dD}{2}\cos\overline{\frac{\pi}{3}+\theta} + t_5\frac{dD}{2}\cos\overline{\frac{\pi}{3}-\theta} + t_6\frac{dD}{2}\cos\theta\Big)$$

$$= \frac{r\pi D^3dmc}{48(1-\cos\alpha)}\Big[(d\cos\theta - \cos\alpha)\cos\theta + \left(d\cos\overline{\frac{\pi}{3}-\theta} - \cos\alpha\right)\cos\overline{\frac{\pi}{3}-\theta} + \left(d\cos\overline{\frac{\pi}{3}+\theta} - \cos\alpha\right)\cos\overline{\frac{\pi}{3}+\theta}$$

$$+ \left(d \cos \overline{\frac{\pi}{3} + \theta} + \cos \alpha\right) \cos \overline{\frac{\pi}{3} + \theta}$$

$$+ \left(d \cos \overline{\frac{\pi}{3} - \theta} + \cos \alpha\right) \cos \overline{\frac{\pi}{3} - \theta}$$

$$+ (d \cos \theta + \cos \alpha) \cos \theta \Big]$$

$$= \frac{r\pi D^3 dmc}{24(1 - \cos \alpha)} \left[d \cos^2 \theta + d \cos^2 \overline{\frac{\pi}{3} - \theta}\right.$$

$$\left. + d \cos^2 \overline{\frac{\pi}{3} + \theta}\right]$$

$$= \frac{r\pi D^3 d^2 mc}{48(1 - \cos \alpha)} \left(3 + \cos 2\theta + \cos \overline{\frac{2\pi}{3} - 2\theta}\right.$$

$$\left. + \cos \overline{\frac{2\pi}{3} + 2\theta}\right)$$

$$= \frac{r\pi D^3 d^2 mc}{16(1 - \cos \alpha)}$$

$$= \frac{r\pi D^3 d^2 mc}{16(1 + \sin \phi)} \quad . \quad . \quad . \quad . \quad (4)$$

Expressions (3) and (4) are both independent of the position and number of the bars in the group provided that there is an even number. This is only true if the modular ratio m, and not $m - 1$, is used in considering compressive stresses in the bars.

Since the total compressive force less the total tensile force in the section is equal to the load, expressions (1) and (3) together equal W, that is,

$$W = \frac{D^2 c Y}{4(1 + \sin \phi)} + \frac{r\pi D^2 mc \sin \phi}{4(1 + \sin \phi)}.$$

Therefore

$$W = \frac{D^2 c(Y + r\pi m \sin \phi)}{4(1 + \sin \phi)} \quad . \quad (5)$$

From (5),

$$c = \frac{4W(1 + \sin \phi)}{D^2(Y + r\pi m \sin \phi)} \quad . \quad (5a)$$

Since the sum of the moments about the centre of the compressive forces added to the moments of the tensile forces is equal to M, expressions (2) and (4) together are equal to WeD, that is

$$WeD = \frac{D^3 cX}{96(1 + \sin \phi)} + \frac{r\pi D^3 d^2 mc}{16(1 + \sin \phi)}$$

and

$$We = \frac{D^2 c\left(\frac{X}{24} + \frac{r\pi m d^2}{4}\right)}{4(1 + \sin \phi)} \quad . \quad (6)$$

Substituting the value of W from equation (5) in equation (6)

$$\frac{D^2 c(Y + r\pi m \sin \phi)}{4(1 + \sin \phi)} \times e = \frac{D^2 c\left(\frac{X}{24} + \frac{r\pi m d^2}{4}\right)}{4(1 + \sin \phi)}.$$

$$Y + r\pi m \sin \phi = \frac{X}{24e} + L \quad . \quad (7)$$

The curves in *Fig. 2* represent the left-hand side of equation (7) for values of r between its normal permissible limits of 0·008 and 0·08. The right-hand side of the equation is represented by a line plotted on the graphs for various values of ϕ. The co-ordinates of the point of intersection of the curve and the line give the solution of equation (7) for values ϕ on the horizontal axis and $Y + r\pi m \sin \phi$ on the vertical axis; these may be substituted in equation (5a). The maximum stress in the steel is found from the stress-variation diagram.

Prestressed Pile 192 ft. Long.

A PRESTRESSED concrete hollow pile 192 ft. long, 3 ft. outside diameter, and with a wall 4 in. thick has been driven in the Gulf of Mexico. The pile contained eight prestressing cables each comprising six wires, which were tensioned to 150,000 lb. per square inch and bonded to the concrete by the pre-tensioning method. The pile contained 9½ lb. of prestressing steel and 5 lb. of untensioned steel per foot. The pile passed through 37 ft. of water and penetrated 22 ft. into the mud under its own weight. It was driven by a single-acting hammer with a 12,500-lb. ram having a stroke adjustable to 24 in. or 39 in.; 4600 blows with a 24-in. stroke and 1000 blows with a 39-in. stroke were used. No difficulty was experienced in driving the pile with extended leaders on a frame used for driving shorter piles. This type of pile is patented by the Raymond Concrete Pile Company.

A Prestressed Bridge in the U.S.A.

PRECAST BEAMS WITH BONDED WIRES CAST ON THE SITE.

A PRESTRESSED precast concrete bridge now being built at Roseville, Ohio, U.S.A., is 220 ft. long and comprises five spans of 44 ft. The construction is unusual in that the beams are being cast on the site on a long-line process. The following description of the bridge is abstracted from the U.S.A. journal, "Engineering News-Record."

Each of the five spans consists of nine precast T-beams 30-in. deep and a cast-in-place deck slab bonded to the beams (*Fig.* 1). Beams were chosen in preference to a thick slab because the bridge is on a 45-deg. skew and because head-room is not an important consideration. The span-depth ratio of the beams is 18 to 1. To reduce the amount of shuttering required for the deck, the beams are placed with the top flanges 8 in. apart. Concrete is cast to a depth of at least 4½ in. over the beams and also fills the spaces between the top flanges, making the total depth of the deck 7½ in. Dowels extend from the sides and top of the flanges into the deck to ensure transfer of the shearing forces between the precast and cast-in-place components (*Fig.* 2).

The prestressing force is applied to each beam by thirty-two ¼-in. diameter cables, each composed of seven strands with an area of 0·0352 sq. in. The cables have an ultimate tensile strength of 260,000 lb. per square inch and were tensioned to 160,000 lb. per square inch. They are placed in the bottom flange and transfer the prestressing force to the concrete by bond. It is assumed that when the tensioning force is removed the stress is about 147,000 lb. per square inch and that the effective stress will be 133,000 lb. per square inch after shrinkage and creep of the concrete have taken place. The stress in the concrete in the bottom flange is 2000 lb. per square inch when the tensioning force is removed and 1600 lb. per square inch ultimately. The crushing strength of the concrete is at least 5000 lb. per square inch when the tensioning force is removed. The prestressing force may create tensile stresses in the top flange, but these are limited to half the modulus of rupture of the concrete. Despite the low tensile stresses, nominal reinforcement is incorporated in the top flange. Under the design load there will be no tensile stresses in the bottom flange and at the same time the compressive stress in the deck or in the top flanges of the beams is less than 600 lb. per square inch. Lower strength concrete may therefore be used for the deck than for the beams. The concrete in the deck slab has a crushing strength of at least 4000 lb. per square inch at 28 days, with a slump not exceeding 2½ in. The leaner mixture is expected to reduce shrinkage of the deck. The maximum shearing stress in a beam will not exceed 200 lb. per square inch and nominal stirrups only, comprising one ⅜-in. bar every 12 in., are used.

Fig. 1.—Beams in Position ready for Concreting the Deck.

The beams in each span are prestressed transversely with four 0·6-in. diameter 7-strand cables, with an area of 0·215 sq. in. These are tensioned to 125,000 lb. per square inch and extend through two cast-in-place diaphragms between the beams, and transfer the prestressing force through end anchorages. The diaphragms are parallel to the bridge piers and abutments.

Fig. 3.—An Abutment.

Casting the Beams.

The casting yard is a concrete road 16 ft. wide at the site and, although the road curves, there is sufficient length for a stressing bed 145 ft. long. Two parallel lines are used in each of which three beams can be cast. The procedure is (1) Tension the cables; (2) Place the shutters; (3) Cast the concrete; (4) When the concrete has attained a strength of at least 5000 lb. per square inch, cut the wires between the beams with a flame; (5) Move the beams into place on to the piers and abutments.

To resist the tensioning force of 160,000 lb. applied to the 32 cables in each beam, concrete abutments, 4 ft. high by 4 ft. thick, were built across the road at both ends of the stressing bed (*Fig.* 3). They transmit the horizontal thrust to the road slab, and overturning is resisted by five piles.

The cables are prepared on a bench 145 ft. long. Each cable is about 600 ft. long, that is somewhat longer than four times the length of the stressing bed. Sleeves for anchoring a cable after tensioning are pressed on the free ends with an hydraulic tool. The cable is looped to form four lengths of about 150 ft. each. The looped ends of one cable are inserted in slots in one face of a steel block 4 in. thick. A similar cable is then placed in slots in the opposite face of the block. At the other end the free ends of the cables are attached

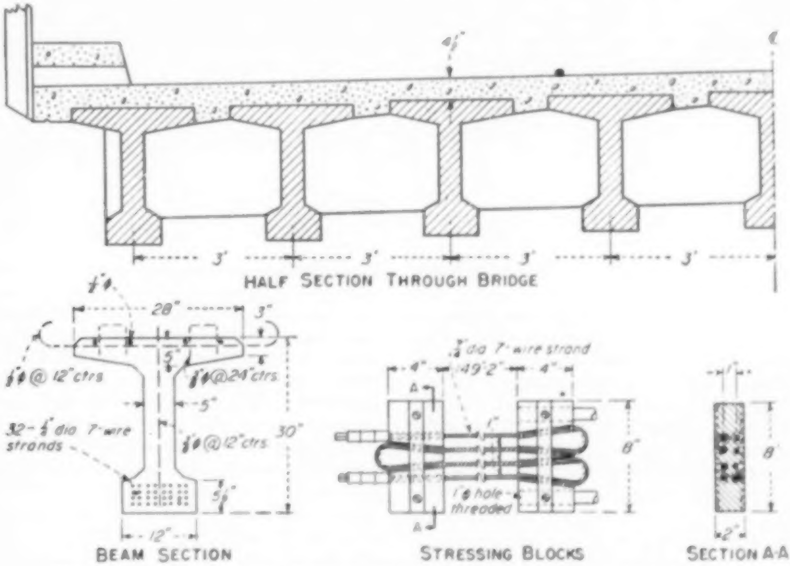


Fig. 2.—Details of the Beams and the Temporary Anchor-blocks.

to a block by the anchoring sleeves and the looped end is inserted in a slot (*Fig.* 2).

The blocks are used to tension eight lengths of cable simultaneously, thus reducing the number of temporary anchorages from 64 to 16 per production line.

After the cables are assembled they are moved to the stressing bed, the blocks being held in position by steel guides in the abutments. Pulling rods, 1 in. diameter, attached to each anchorage block, are then fastened to a 30-tons jack and the strands stretched until a gauge indicates that the required force has been applied. The steel has then elongated 12¼ in. Steel shims are inserted between the anchorage blocks and a steel bearing-plate in the abutment to maintain the extension.

Three sets of plywood shutters, one for each beam in a production line, were designed to be used fifteen times. They are in short lengths for easy handling, and are put in place rapidly, after which all the reinforcement bars, dowels, and stirrups are inserted (*Fig.* 4).

The concrete contains 705 lb. of air-entraining cement per cubic yard with an addition of one per cent. of calcium chloride. The strength at seven days exceeds 5000 lb. per square inch. The shutters are stripped within two days, and the concrete is cured for five to seven days. The wires are cut a few cables at a time throughout the length of the line, to reduce movement of the beams when the force is released.

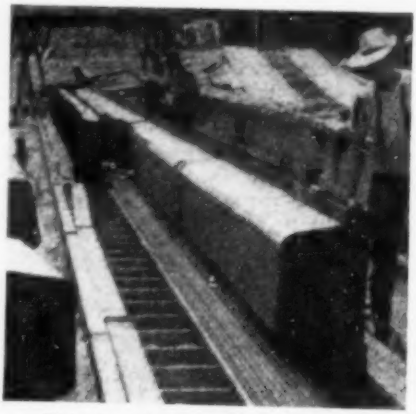


Fig. 4.—Shuttering for Beams.

Transverse Prestressing.

The cables for transverse prestressing are passed through holes in the beams, and wrapped with paper to prevent bond with the cast-in-place concrete of the diaphragm. The diaphragms and the deck are concreted simultaneously and, when the concrete has attained sufficient strength, the transverse cables are tensioned and anchored at their ends by tightening a nut on swaged fittings.

The bridge was designed and constructed by Messrs. Wander and Mason, Inc., of Worthington, Ohio, and the design was checked by Messrs. Alden E. Stilson and Associates, consulting engineers for the Board of Commissioners of Muskingum County, Ohio.

"Popcorn" Concrete.

In this country concrete containing no sand is known by the somewhat unfortunate term "no-fines" concrete. In Holland, where it originated early this century, it was known as "Korelbeton" because it resembled coral. In a paper read at a recent convention of the American Concrete Institute, Professor George W. Washa, chairman of the Lightweight Concrete Committee of the Institute, describes it as popcorn concrete. It is a pity that when the process was copied from Holland it should have been thought necessary to use a different term, for coral concrete is a better and more properly descriptive term than either no-fines or popcorn.

Tests on Lightweight Prestressed Beams.

TESTS on prestressed concrete made with expanded shale have been made by the Carter-Waters Corporation of Kansas City and Prestressing, Inc., of San Antonio, Texas, U.S.A. The beams were 32 days old when they were tested.

The first tests were made on the beam shown in *Fig. 1*. The concrete consisted of 635 lb. of cement, 1·1 cu. yd. of aggregate, and 50·5 U.S. gallons (42 Imperial gallons) of water, which when mixed together made one cubic yard of concrete with a density of 105 lb. per cubic foot and a compressive strength (tested as cylinders) of 6,000 lb. per square inch at 28 days. A total prestressing force of 113,500 lb. was applied by sixteen 0·25-in. diameter wires with a yield strength of 183,000 lb. per square inch. The beam was designed to carry a load of 19,500 lb.

For the second series of tests two beams were made, each similar to that used in the first test except that the thickness of the webs was 5 in. and the width of the flanges 11½ in. The prestressing force in each beam was 127,800 lb. and the design load 26,600 lb. In one beam the cables were bonded with cement grout and the other had unbonded cables.

The first tests consisted of applying loads at third points, measuring the deflections, and taking strain-gauge readings. The five loads applied were (1) 70 per cent. of the design load, (2) 125 per cent. of the design load, (3) loading the beam until it cracked, (4) loading the beam until it cracked across the whole of the middle third of its length, and (5) loading until the beam broke. The beam was allowed time to recover from one application of the load before the next was made.

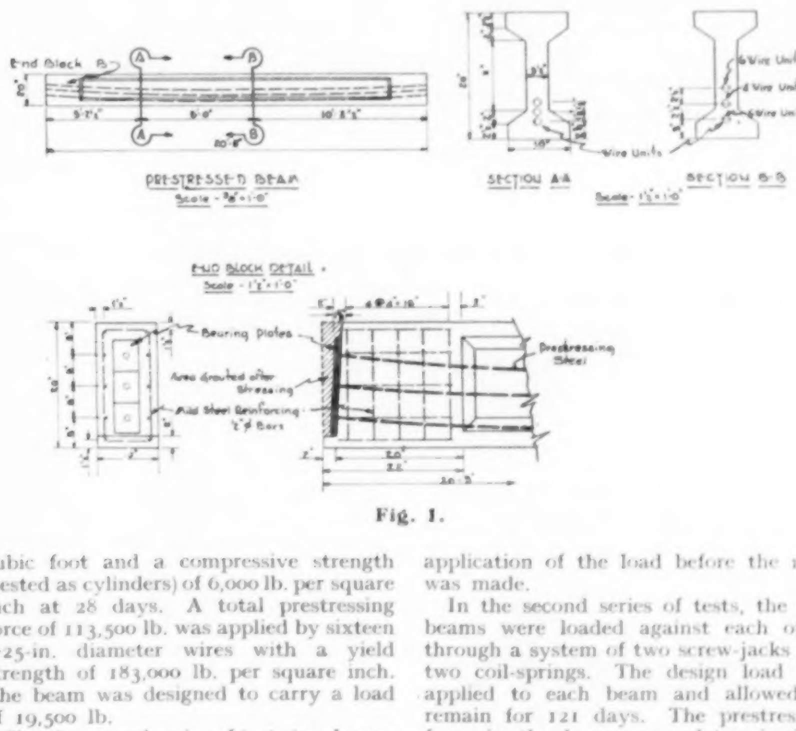


Fig. 1.

In the second series of tests, the two beams were loaded against each other through a system of two screw-jacks and two coil-springs. The design load was applied to each beam and allowed to remain for 121 days. The prestressing force in the beams was determined at intervals during this period. After this test each beam was tested separately to destruction.

From the tests it is concluded that

(1) The strength of the concrete was adequate for prestressing.

(2) The modulus of elasticity of the concrete was sufficiently high for prestressing. Reductions of the modulus of elasticity when the beams were loaded for a long period were not excessive, and when the load was removed and then reapplied the beams recovered a sufficiently high modulus of elasticity. The moduli of elasticity were as follows. In the first tests: At the time of loading, $3{\cdot}24 \times 10^6$ lb. per square inch; after a load of 70 per cent. of the design live load had been applied for 46 hours, $2{\cdot}85 \times 10^6$ lb. per square inch. After a load of $1{\cdot}25$ times the design load had been removed the modulus apparently recovered to $3{\cdot}15 \times 10^6$ lb. per square inch. In the second tests: At the time of loading, $3{\cdot}38 \times 10^6$ lb. per square inch; after two days of loading, $2{\cdot}75 \times 10^6$ lb. per square inch; after 121 days, $1{\cdot}73 \times 10^6$ lb. per square inch; when the load was removed the modulus recovered to $2{\cdot}31 \times 10^6$ lb. per square inch.

(3) The losses of the prestressing forces due to shrinkage and creep were not excessive and an allowance of 25 per cent. loss of the initial prestressing force is thought to be satisfactory. The actual losses were calculated from the results of the 121 days' test as 19 per cent. for the beam with ungrouted cables and 17 per cent. for the beam with bonded cables.

(4) The beam with bonded wires broke when the load was 59,900 lb. and the beam with unbonded wires at 44,000 lb. The load at which cracks first appeared was slightly higher in the case of the bonded wires.

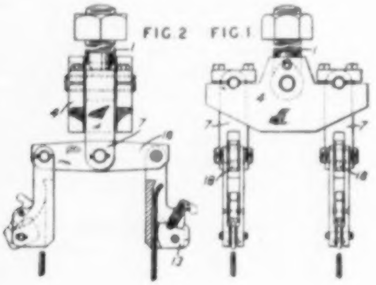
(5) Lightweight concrete behaves elastically for loads of up to the design load.

The foregoing notes are based on an article by Mr. Fred E. Koebel in the Journal of the American Concrete Institute for March, 1954.

Patent Relating to Concrete.

A Device for Tensioning Wires.

In a device (*Figs.* 1 and 2) for tensioning a number of wires simultaneously, a force is applied to a primary bridge member (4) at a point between its ends and thence from each end of the member to a point between the ends of each of a pair of sub-bridge members (18), and from the ends of each member (18) to the separate wires. The member (4) is pivotally connected at its centre to a bolt (1) and at each end pivotally supports a link (7) which is pivotally connected to the centre of one of the members (18). To each end of each member an anchor for a wire is pivotally connected. Each anchor has a spring-loaded serrated jaw (13) which may be locked in an open position. A force applied to the bolt (1), by an hydraulic jack for example, is applied equally to the wires held by the four anchors. To anchor the wires in the jaws a pin is used to lock the bolt (1) to the bridge (4) and wedges prevent movement of members (18).



The device is modified for use with three wires by arranging the members (18) parallel to the bridge (4), connecting the links (7) so that the outside arm of each member (18) is twice the length of the inside arm, and joining the two inside arms to a common anchor. Unequal forces may be applied to the wires by pivoting the bridges, and the device may be modified for tensioning more than four wires.—No. 657,794. Minister of Works. February 4, 1949.

[Publication of Patent Specifications is in arrears due to the war.]

Eccentrically-loaded Columns.

The results of an investigation of the behaviour of eccentrically-loaded reinforced concrete columns are given in a report entitled "A Study of Combined Bending and Axial Load in Reinforced Concrete Members," by Eivind Hognestad (University of Illinois Engineering Experiment Station, Bulletin No. 399). A total of 120 columns were loaded to failure in short-time tests, the period between the first and last increments of load being about one hour. The columns were divided into four groups, each of thirty columns. The sizes of the columns and the amounts of reinforcement are shown supports of the columns except for the concentrically-loaded helically-reinforced columns which were tested with flat ends. All the columns were cast in a vertical position and they all failed in the upper half; this was considered to be due to variations in the strength of the concrete from the bottom to the top of a column due to differences in the compaction. Electrical-resistance strain-gauges were used to measure the strains in the reinforcement and near the surface of the concrete; the lateral deflections of the columns in the plane of the eccentricity were also measured.

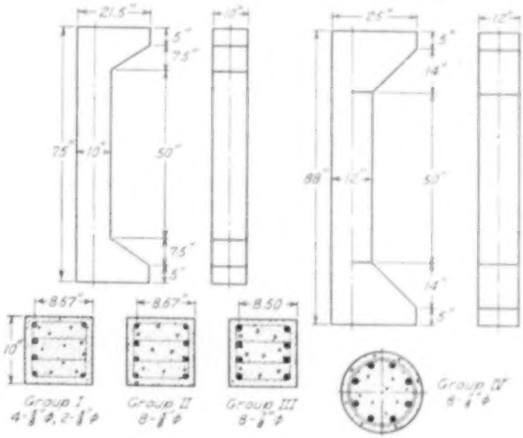


Fig. 1.—Details of Columns.

in *Fig.* 1. Within each group the strength of the concrete at 28 days varied from 1500 lb. to 5500 lb. per square inch. The longitudinal reinforcement comprised deformed bars with an average yield stress of about 43,500 lb. per square inch, the rectangular hoops were ¼-in. bars with a yield stress of about 60,000 lb. per square inch, and the helical binding was hard drawn wire of about ¼ in. diameter with an average ultimate tensile stress of about 115,000 lb. per square inch. The columns were tested in a machine with a capacity of 3,000,000 lb. The eccentricity of the load was varied from 0 to 1¼ times the lateral dimension of a column and the load was applied through "knife-edges" at the top and bottom

Loads Causing Failure of Rectangular Columns.

The ultimate loads on the columns of groups I, II and III are shown in *Figs.* 2, 3, and 4, which show also the ultimate loads calculated by the theories due to Mr. Whitney, Mr. Jensen, and the author.* The eccentricities shown on these graphs are the initial eccentricities of the loads plus the deflections at failure measured with respect to the knife-edges through which the loads were applied.

In agreement with the findings of earlier investigators, it was found that the ultimate loads for columns loaded concentrically through knife-edges were 10 to 15 per

* See references on page 308.

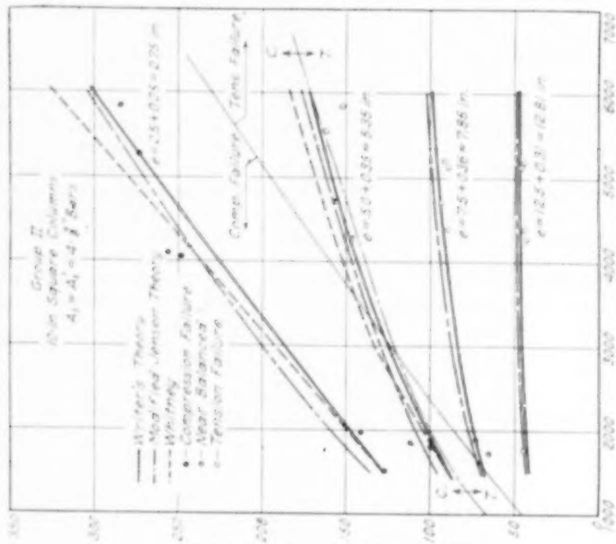


Fig. 3.—Loads at Failure, Group II.

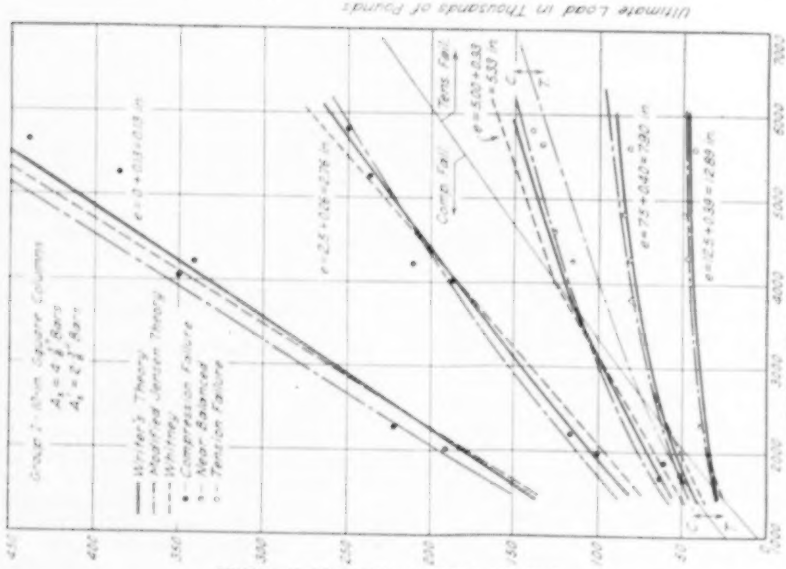


Fig. 2.—Loads at Failure, Group I.

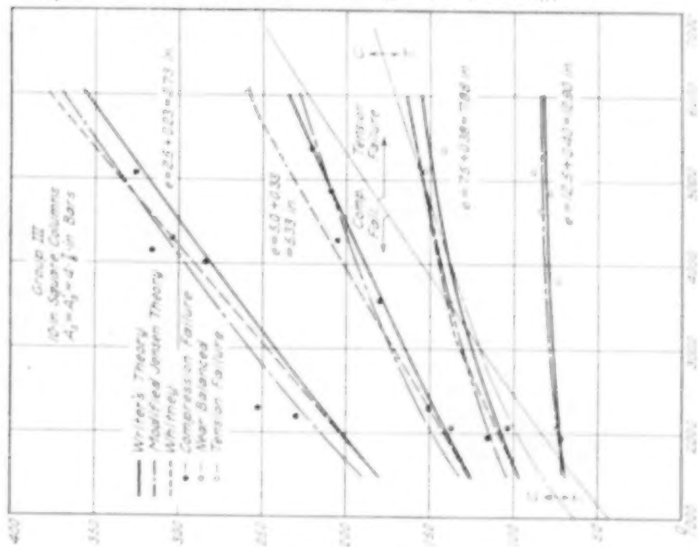


Fig. 4.—Loads at Failure, Group III.

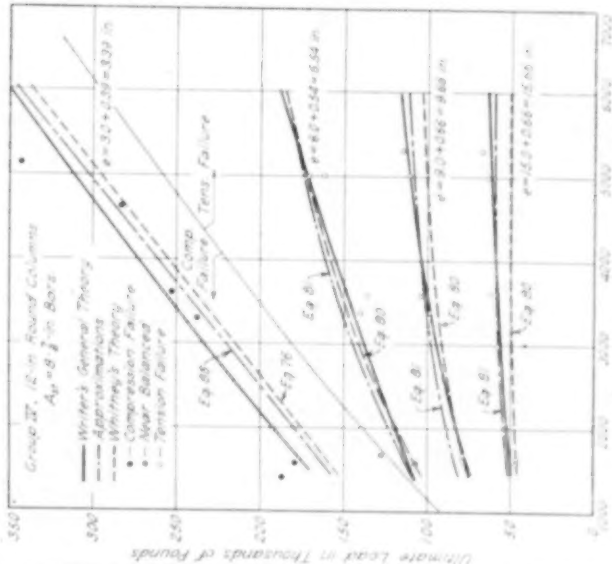


Fig. 5.—Loads at Failure, Group IV.

cent. lower than for similar columns tested with flat ends. This is explained as being due to the difficulty of obtaining a truly concentric application of load through knife-edges. The failure of all eccentrically-loaded rectangular columns was caused by crushing of the concrete at an ultimate strain of about 0·0038. After such crushing had taken place, the reinforcement in compression buckled between the ties, and the strength of the columns was thereby reduced very considerably. The reinforcement in compression in most cases yielded before failure of the concrete. Preceding the failure of the concrete, all eccentrically-loaded columns were in a state of equilibrium with the applied load such that there were considerable increases in deformation for very small additions of load. Columns which failed in tension developed much larger deflections before final crushing of the concrete than did the columns that failed in compression. A fairly linear distribution of strains across the column existed from the smallest loads to failure.

Loads Causing Failure of Helically-reinforced Columns.

The loads causing failure of the helically-reinforced columns are shown in *Fig.* 5 in the same manner as for the rectangular columns.

The concentrically-loaded columns were loaded with flat ends. Their general behaviour was in agreement with the findings of earlier investigators. After the cover spalled the columns continued to deform; by the action of the helical reinforcement, a higher load was carried than that causing spalling.

The ultimate load of the eccentrically-loaded columns, which were loaded through knife-edges, was, however, reached at failure of the concrete cover. After failure of the cover, all the eccentrically-loaded columns developed extremely large deflections without serious decreases in load-carrying capacity, since the binding prevented buckling of the longitudinal reinforcement and crushing of the concrete core. Regardless of the eccentricity of load, the columns were able to carry fairly high loads and deform very considerably.

Calculation of the Loads Causing Failure.

The Bulletin contains an historical review of the methods of calculating the ultimate load which a reinforced concrete member will resist, and in particular the author compares the methods due to Mr. Whitney and Mr. Jensen with the results of the tests, on the basis of which the following theory was developed.

The distribution of stress and strain across a rectangular member was assumed to be as shown in *Fig.* 6(*a*), in which the notation not obvious from the diagram is: f_c', compressive strength of cylinders 6 in. diameter by 12 in. high; f_{yp}', yield stress of the reinforcement in compression; A_s', area of steel in compression; C', total compressive force in the steel; C, total compressive force in the concrete; f_c'', compressive strength in bending of concrete (assumed to be $0{\cdot}85\ f_c'$); b, width of the column; k_1 and k_2, constants defining the area of the stress-diagram and its centre of gravity; ε_u ultimate strain of the concrete in bending.

From *Fig.* 6(*a*), and calculating moments about the steel in tension,

$$Pe' = k_1bcf_c''(d-k_2c) + A_s'f_{yp}'d' \quad . \quad (1)$$

Also

$$P = k_1bcf_c'' + A_s'f_{yp}' - A_sf_s \quad . \quad . \quad (2)$$

and

$$f_s = \varepsilon_uE_s\frac{d-c}{c} < f_{yp} \quad . \quad . \quad . \quad . \quad (3)$$

The notation is as previously defined, except for E_s which is the modulus of elasticity of the steel and f_{yp} the yield stress of the steel in tension.

Failure in tension starts by yielding of the steel in tension; this produces a movement of the neutral axis towards the edge of the section which is compressed, and the ultimate load is reached when the compressed concrete crushes. At failure $f_s = f_{yp}$, and from (1) and (2)

$$P = f_c''bd\left[q'-q+\frac{1}{2\alpha}\left\{-\left(\frac{e'}{d}-1\right)+\sqrt{\left(\frac{e'}{d}-1\right)^2+4\alpha\left[q'\frac{d'}{d}+\frac{e'}{d}(q-q')\right]}\right\}\right] \quad (4)$$

in which $q' = \dfrac{A_s'f_{yp}'}{bdf_c''}$,

$$q = \frac{A_sf_{yp}}{bdf_c''}, \text{ and } \alpha = \frac{k_2}{k_1}.$$

Equation (4) may be simplified as follows for symmetrical reinforcement, $q' = q$:

$$P = \frac{f_c''bd}{2\alpha}\left[-\left(\frac{e'}{d}-1\right) + \sqrt{\left(\frac{e'}{d}-1\right)^2 + 4\alpha q\frac{d'}{d}}\right] \quad . \quad . \quad (5)$$

When there is no compressive reinforcement, $q' = 0$, and

$$P = f_c''bd\left[-q + \frac{1}{2\alpha}\left\{-\left(\frac{e'}{d}-1\right) + \sqrt{\left(\frac{e'}{d}-1\right)^2 + 4\alpha q\frac{e'}{d}}\right\}\right] \quad . \quad . \quad (6)$$

The values of k_1 and k_2 vary with the compressive strength of the concrete but their ratio α is very nearly constant and has a value of 0·55.

Failure in compression will take place if the compressed concrete fails before the steel in tension yields. The stress in the tensile steel must therefore be calculated from (3). The calculation of P may then be made from (1) and (2), and as their solution involves a cubic equation it may best be carried out by iterative or graphical methods.

The limiting condition at failure in which the steel in tension yields as the concrete crushes (known as the balanced condition) can be defined in terms of $k = \frac{c}{d}$, indicating the position of the neutral axis at failure. For the balanced condition this ratio is $k_b = \frac{\epsilon_u}{\epsilon_u + \frac{f_{yp}}{E_s}}$.

Failure in tension will occur if $k < k_b$, and failure in compression will result if $k > k_b$. By introducing $k_b = \frac{c_b}{d}$ into (1) and (2) the eccentricity causing balanced failure and the corresponding ultimate load may be calculated.

If the eccentricity is small the neutral axis will be outside the section, and from

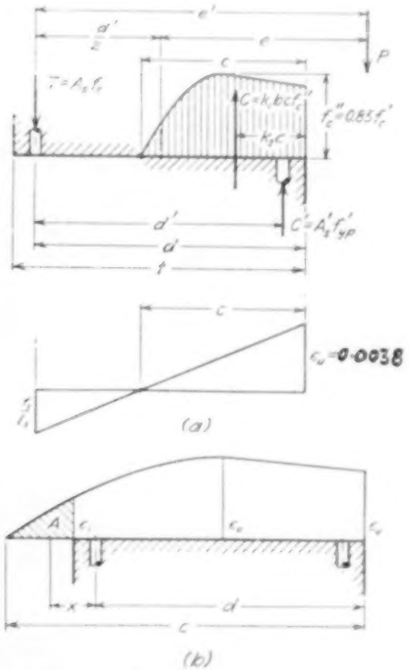


Fig. 6.—Rectangular Columns.

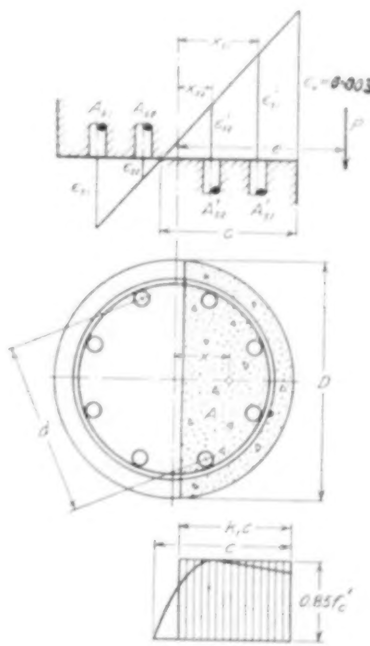


Fig. 7.—Circular Columns.

Fig. 6(*b*) equations (1) and (2) are modified to

$$Pe' = k_1 f_c'' bc(d - k_2 c) + A_s' d' f_{yp}' + Abx \quad (1a)$$

$$P = k_1 f_c'' bc + A_s' f_{yp}' - A_s f_s - Ab \quad . \quad (2a)$$

in which A and x may be found by geometry.

In the case of circular sections (*Fig.* 7) an equivalent uniform distribution of stress is assumed over a segment with a rise $k_1 c$, and the author assumed that the values of k_1 for a rectangular section could be used as an approximation. The resultant compressive force in the concrete was assumed to act at the centroid of the segment, the distance x from the centre of the section to the centroid of the section being expressed approximately, according to Mr. Whitney, as

$$\frac{x}{D} = \frac{2}{3\pi} + 0{\cdot}293\left(\frac{\pi}{4} - \frac{2A}{D^2}\right),$$

in which A is the area of the segment.

The following equations apply to failures by both tension and compression and, from *Fig.* 7, by the equilibrium of forces,

$$P = 0{\cdot}85 f_c' A + E_s \Sigma A_s \varepsilon_s \quad . \quad . \quad (7)$$

From the equilibrium of moments,

$$P_e = 0{\cdot}85 f_c' A x + E_s \Sigma A_s \varepsilon_s x_s \quad . \quad (8)$$

By geometry,

$$\left.\begin{aligned} \varepsilon_{s1} &= \frac{\varepsilon_u}{c}\left(\frac{D}{2} + x_{s1} - c\right) \\ \varepsilon_{s2} &= \frac{\varepsilon_u}{c}\left(\frac{D}{2} + x_{s2} - c\right) \\ \varepsilon_{s1}' &= \frac{\varepsilon_u}{c}\left(c - \frac{D}{2} + x_{s1}\right) \\ \varepsilon_{s2}' &= \frac{\varepsilon_u}{c}\left(c - \frac{D}{2} + x_{s2}\right) \end{aligned}\right\} \quad . \quad . \quad (9)$$

Because of the assumed trapezoidal distribution of stress to strain for the reinforcement,

$$\varepsilon_s < \frac{f_{yp}}{E_s} \quad . \quad . \quad . \quad . \quad (10)$$

the condition referred to as balanced failure will exist if, at the ultimate load,

$$c = \frac{\varepsilon_u}{\varepsilon_u + \frac{f_{yp}}{E_s}}\left(\frac{D}{2} + x_{s1}\right) \quad . \quad . \quad (11)$$

Graphs plotted from these formulæ are shown in *Figs.* 2 to 5 in relation to the observed ultimate loads for the columns tested.

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Heating Coils in Concrete Floors.

The hot-water heating system installed in the floors of a block of residential flats 14 stories high at Chicago is described by the American Heating, Piping and Air-conditioning Contractors' Association. The system is designed to produce a comfortable temperature inside the building when the outdoor temperature is − 10 deg. Fahr. The coils are of copper tube of ⅜-in. outside diameter, and are placed ¾ in. from the bottom of reinforced concrete floor slabs 7 in. thick. Water is supplied from a tank on the roof at a temperature of 175 deg. Fahr and returned at 145 deg. Fahr.; the average temperature in the coils is 160 deg. Fahr. About two-thirds of the heat is directed downwards through the ceiling and the remainder upwards through the floor above. It is claimed that this method is cheaper than other means of heating blocks of residential flats.

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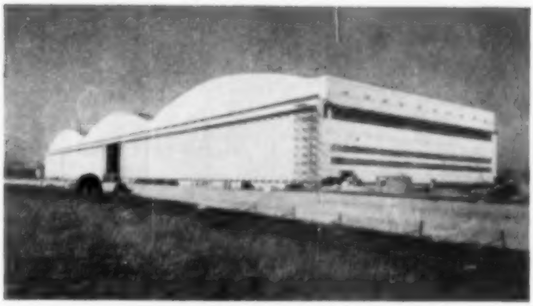

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New System of Prestressed Floor Construction.

THE first floor of a two-story extension at the Plastool factory at Foots Cray, Kent, is designed to carry fairly heavy machinery which is liable to excessive wear as a result of vibration, and it was required to distribute the concentrated loads in both directions with as little deflection as possible. The area of the floor is 590 sq. yd., and the design load 200 lb. per square foot plus 18 lb. per square foot for finishes. A Raphcon prestressed precast floor, as shown in the accompanying illustrations, was used. These floors are made with either hollow units the wires in which are pre-tensioned, or hollow reinforced units that are prestressed by the post-tensioning method. The precast units are made on a collapsible core which can be adapted to form units from 5 in. to 10 in. deep by 12 in. to 15 in. wide, suitable for spans of 16 ft. to 40 ft., preferably arranged in two, three, or four bays to allow for continuous spans. The units are prestressed longitudinally to form a continuous slab with bays of 16 ft. 4 in., 11 ft. 6 in., and 16 ft. 4 in. (see *Figs.* 1 and 2).

In constructing this type of floor three or five short lightly-reinforced units are arranged in line in each span, supported on temporary props, with gaps of 4 in. to 6 in. between their ends (*Fig.* 2). Prestressing wires are placed on either side on the units, and steel reinforcement placed transversely over the prestressing wires at the gaps and under the wires at the main supports.

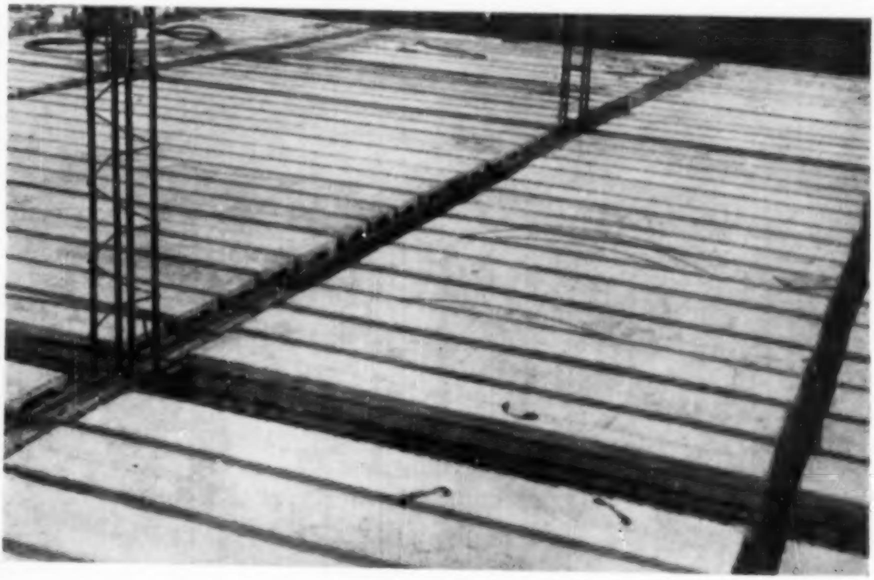


Fig. 1.

With a smaller central span, as in this example, the load on the outer bays can cause a negative bending moment on the central span, which would produce tensile stresses in the top. Some of the prestressing wires are therefore arranged in both the top and bottom of the central span, but they are all in the bottom of the outer bays in order to produce compression to counteract any tension caused by any combination of loading. The gaps

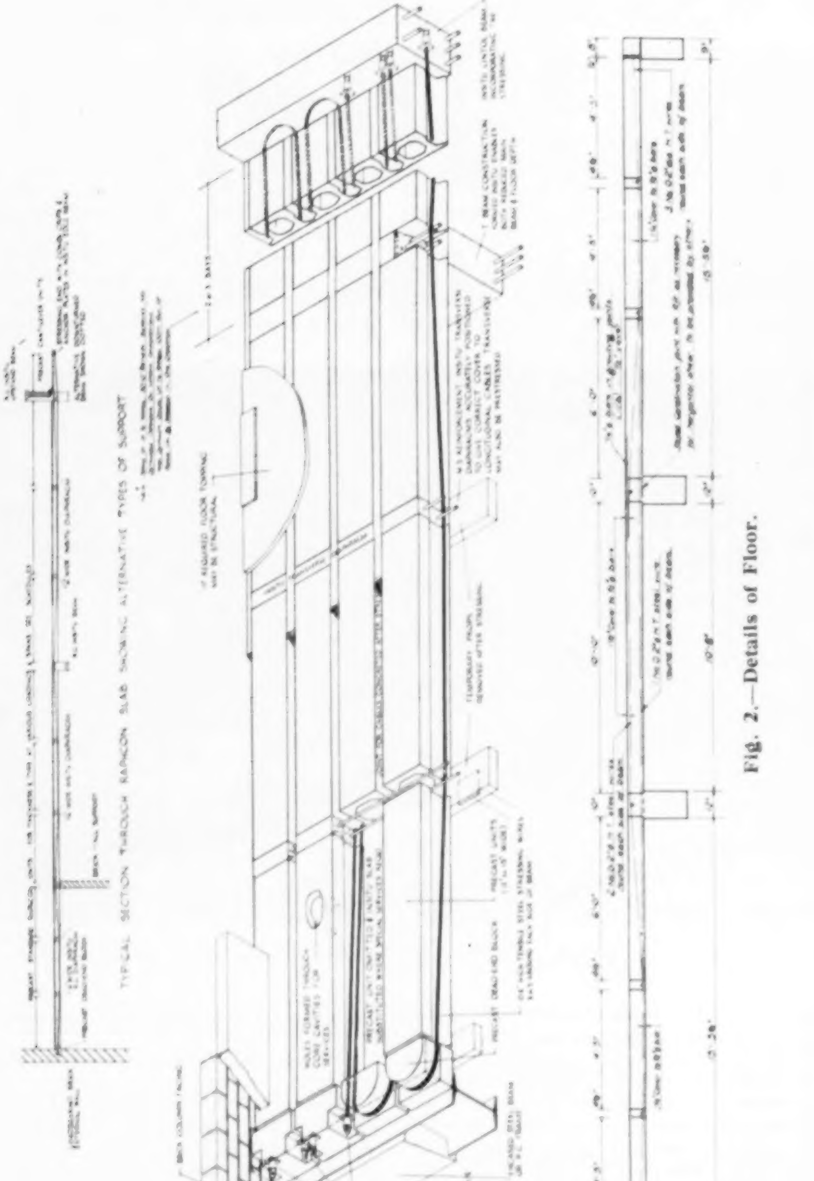
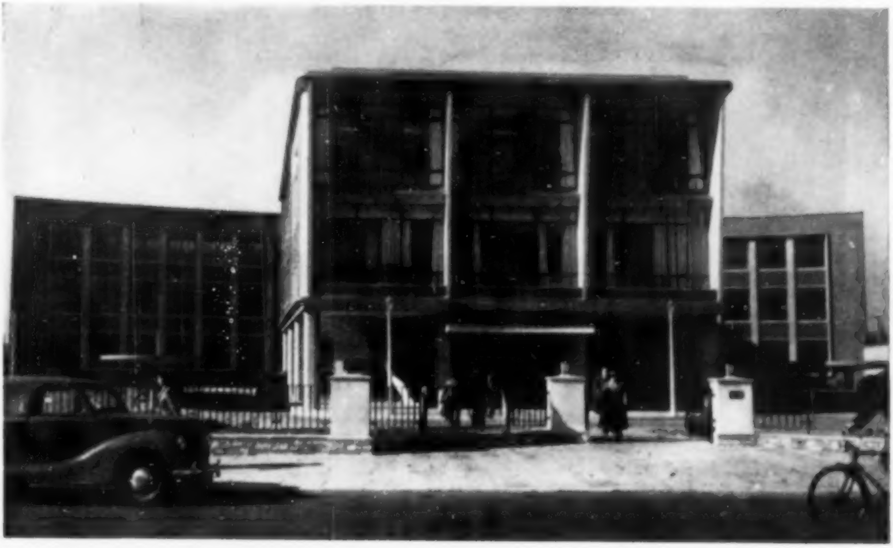


Fig. 2.—Details of Floor.



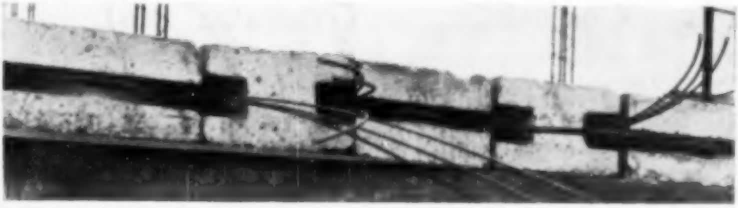


Fig. 3.—Anchor Blocks.

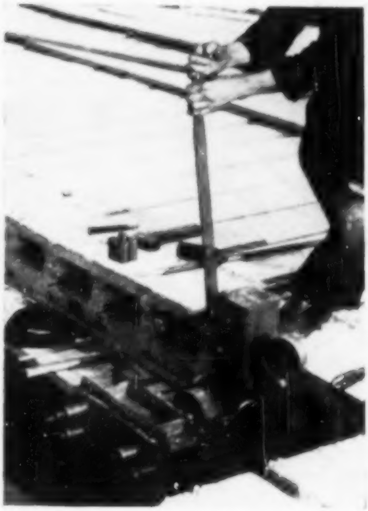


Fig. 4.—Tensioning the Wires.

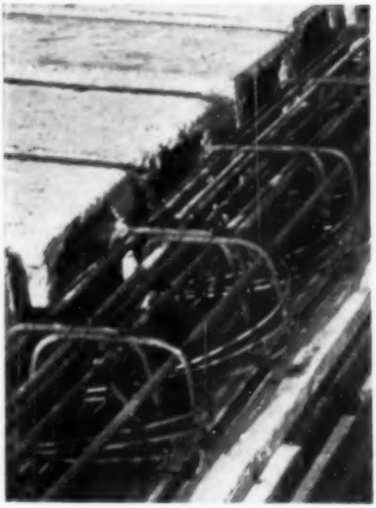


Fig. 5.—Cables anchored to Reinforcement of Edge Beam.

between the units are then concreted to form continuous beams.

At one end of the floor the wires pass around a semi-circular anchorage (*Fig.* 3) similar to that used in some German systems, and at the other end they are tensioned by a jack (*Fig.* 4) bearing against a precast reinforced concrete block. The wires may be tensioned from one end or alternate ends. Alternatively the prestressing wires may at the anchorage end pass around the reinforcement of an edge-beam (*Fig.* 5), and be tensioned after the edge-beam has been concreted in situ.

After prestressing, the floor is fully suspended and continuous before the spaces between the beams are concreted ; services can therefore be installed before the beams are concreted together. No topping is required structurally. The floor acts as a monolithic slab over the three bays, and has about one-fifth of the deflection of a simply-supported slab.

The architect is Mr. V. A. Asbridge, A.R.I.B.A., and the contractors the Vibrated Concrete Construction Co., Ltd., who manufactured the units in conjunction with Raphcon, Ltd.

Consolidating Granular Soil.

A METHOD of consolidating sand or gravel soils by wetting and vibration was used in Germany some twenty years ago. It has been used in the United States of America for the past ten years, and is now available in this country. The process is a method of consolidating columns of such soils up to depths of 50 ft. and diameters of 10 ft. and so increase their load-bearing properties.

The apparatus (*Fig.* 1) is suspended from a crane, and the process is as follows (from left to right in *Fig.* 2). (1) The vibrator is started and the bottom jet is opened to saturate the soil for the vibrator to penetrate; (2) With the jet open, the machine is allowed to sink into the soil by its own weight; because of the vibration and upward movement of the water, a "quick" condition is produced which enables the vibrator to descend rapidly. (3) Compaction of the soil surrounding the vibrator takes place and a cone-shaped crater about 3 ft. diameter is formed at ground level. When the machine has reached the desired depth, the water is delivered from the top jets. The amount of water issuing from these openings is much less than that supplied from the bottom jet and its downward flow helps to carry the sand along the sides of the machine to the bottom of the hole. The combined action of the vibration and the downward flow of water compacts the soil and, as compaction takes place, additional sand is shovelled in at the top to compensate for the loss of volume caused by the increased density of the soil. (4) Actual compaction takes place during the intervals between the 1-ft. lifts which are made in returning the machine to the

Fig. 1.—Vibrator Suspended from a Crane.

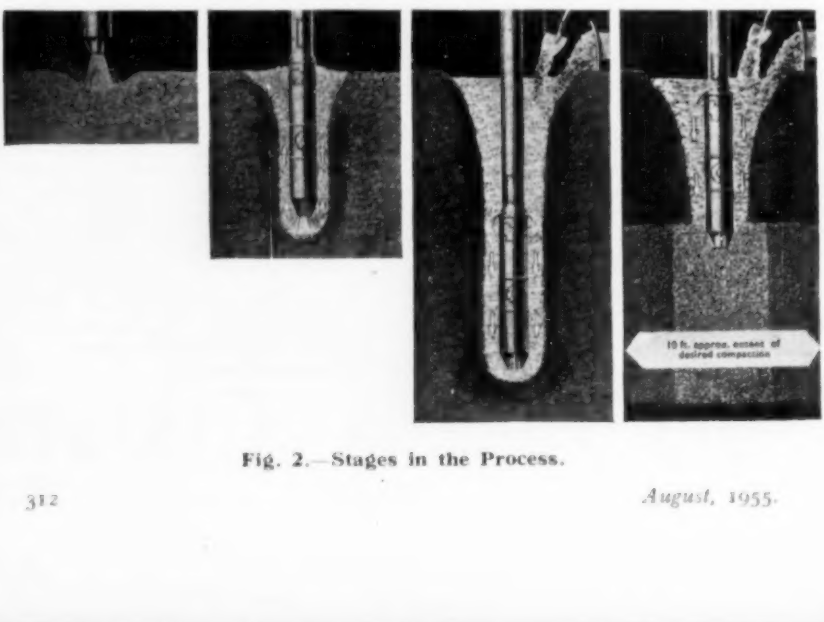


Fig. 2.—Stages in the Process.

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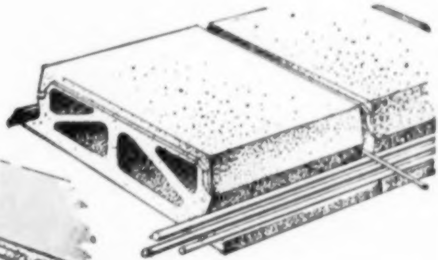

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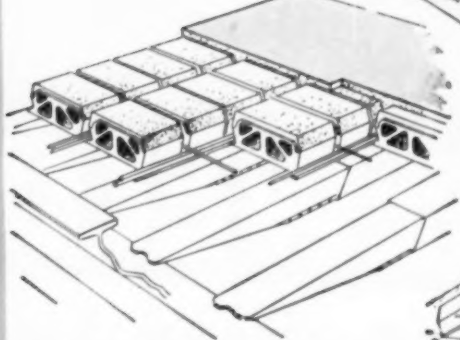

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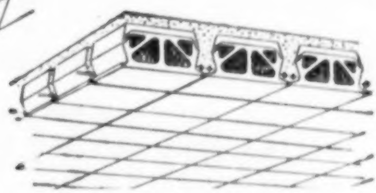

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surface. The vibrator operates at the bottom of the hole until the desired density around the lower part of the machine is attained, and by raising the vibrator step by step and backfilling simultaneously the entire depth of soil is compacted. The machine is used at centres of 6 ft. to 10 ft. apart so that the columns of compacted soil overlap. While the machine is working the density of the soil being compacted is recorded by an electrical device.

The consumption of water is 60 gallons a minute at a pressure of about 70 lb. per square inch. The weight of the apparatus for a penetration of 12 ft. is 2 tons. The licensees for the process in Great Britain and some other countries are Vibrofounds, Ltd., of Southall, Middlesex.

A Tunnel in Scotland.

NEW ROCK-REMOVAL CAR.

THE accompanying illustrations show the method of driving a tunnel at Allt-na-Lairige, Argyll, for the North of Scotland Hydro Electric Board in connection with a new hydro-electric works. The tunnel will convey water a distance of 6600 ft. through a hill. It is 8 ft. high by 6 ft. 6 in. wide and is driven through granite. The small cross section and the nature of the rock made an advance of more than 8 ft. per cycle unlikely, but it was decided to attempt a higher speed. The air pressure used was 115 lb. per square inch at the face, and three Holman "Silver 3" hand drills were used to drill 21 holes in the pattern shown in *Fig.* 1. At the completion of the tunnel each machine had drilled about 30,000 ft. Tungsten-carbide tipped drills of $\frac{7}{8}$-in. hexagonal section with $1\frac{1}{8}$-in. diameter bits were used. To avoid loss of time in changing, drills 8 ft. long were used, and they had an average

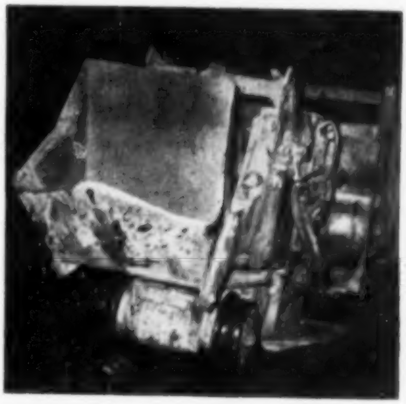

Fig. 2.—The Shovel.

"life" of about 300 ft. The 8-ft. holes were drilled in just over four minutes and the drilling speed (including time for moving from hole to hole) was 18 in. to 20 in. per minute throughout the work. The total drilling time was thus less than 35 minutes per cycle.

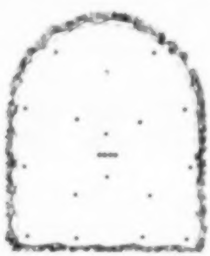

Fig. 1.—Pattern of Drilling.

Broken rock was loaded with an Eimco Model 21 "Rockershovel" (*Fig.* 2). The width of the tunnel prevented any possibility of fast car-changing, and a special shuttle-car was used to ensure continuous loading. The general arrangement of this car is shown in *Fig.* 2 and

Fig. 3.—Arrangement of Rock-removal Car.

Fig. 4.—The Locomotive and Part of the Rock-removal Car.

photographs are given in *Figs.* 4 and 5. The main frame is 70 ft. long and is carried on two 8-wheel bogies. The chassis is supported on stub-axles integral with the frame of the bogie. The floor consists of a steel-slat conveyor-belt on rollers, propelled by hydraulic rams, which move back the loaded rock to make room for more. The rock moved as a rectangular mass about 3 ft. deep between the sides, which are 4 ft. 6 in. apart. The load was

Fig. 5.—Discharging the Rock.

discharged (*Fig.* 5) through the bottom of the car at the rear into tubs of 2 cu. yd. capacity. With the use of this car the time of clearing the rock after each drilling cycle was reduced from 80 minutes to 40 minutes.

The car travelled on a track of 2 ft. gauge with 30-lb. rails, and curves of a minimum radius of about 120 ft. were negotiated. It was pulled by a modified 38-h.p. Diesel locomotive supplied by Messrs. F. C. Hibberd & Co., Ltd., who also collaborated in designing the controls, which could be operated from the loading end of the car 70 ft. away.

Twelve-hour shifts were worked by nine men, comprising a foreman, three drillers, two mechanics, one loader operator, one loco. driver, and one handy man. During the seven days from March 31 last, despite the loss of more than a shift in rectifying minor faults in the car, 67 cycles were completed, and the tunnel was driven 444 ft., which is thought to be a record. The consulting engineers are Messrs. Babtie, Shaw & Morton, and the contractors Messrs. Marples, Ridgway and Partners, Ltd.

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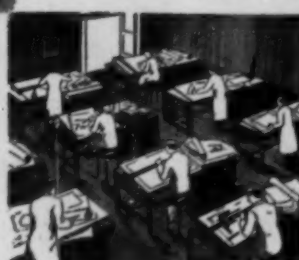

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